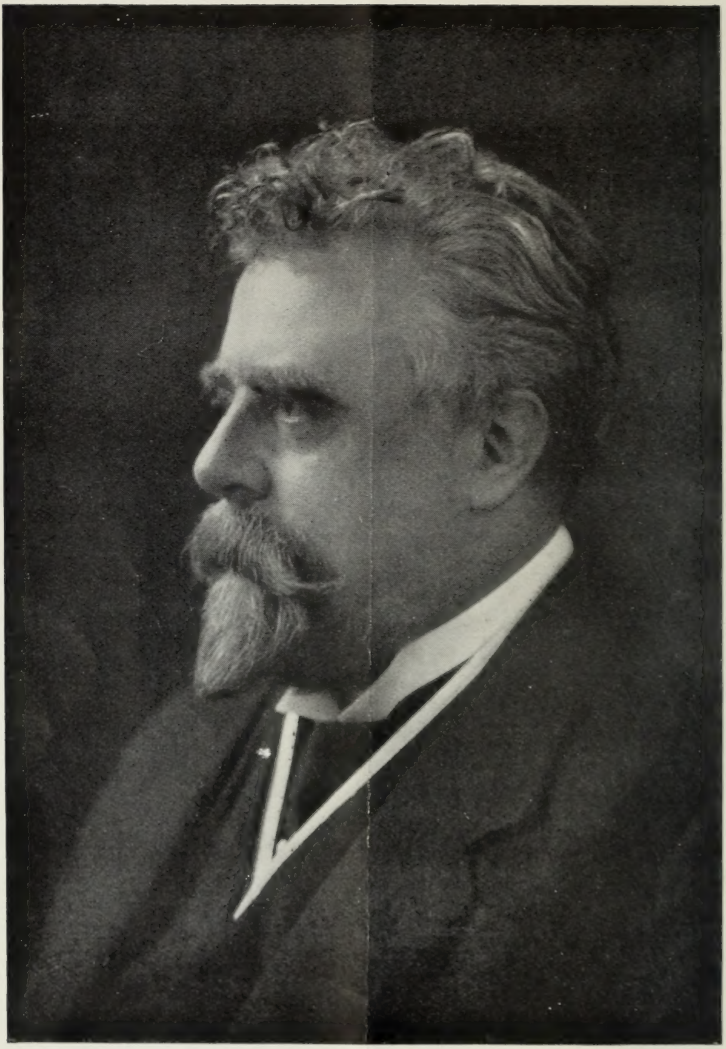
UNIV. OF
TORONTO
LIBRARY

BINDING LIST NOV 1 1924

Digitized by the Internet Archive
in 2011 with funding from
University of Toronto

http://www.archive.org/details/structuralengine12inst

Frontespiece



(*Photo: Elliott & Fry.*)

E. FIANDER ETCHELLS, ESQ.,

A.M.INST.C.E., A.M.I.MECH.E., HON. A.R.I.B.A., ETC.

PRESIDENT OF THE CONCRETE INSTITUTE, 1920-21, 1921-22.

STRUCTURAL ENGINEER.
V.12
(1922)
192837
2.12.24

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *JANUARY, 1922.* **No. I.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

FOREWORD.

The Council here presents to Members of the Concrete Institute the first number of a new "Journal," which they propose shall be issued monthly to the Members during each Session of the Institute.

The last Volume of the Annual Transactions, No. IX, was issued in April, 1921; its delay was due to the abnormal increase in the cost of printing during and after the War.

Volumes X and XI still remain unissued for the same reason.

The Council had to decide whether to continue to issue the back volumes much out of date, which state of affairs might have had to continue for some years; or whether to commence issuing to Members at once, the papers read during the current Session.

They felt that the latter course would be the preferable one to Members. It was also felt that an Institute developing so rapidly in Membership and influence as the Concrete Institute is doing, should issue a monthly up-to-date publication to its Members.

The unissued Volumes, Nos. X and XI, will be issued to Members from time to time.

THE PRESIDENT.

Years before the fateful battle of Trafalgar was fought and won, there flourished in the London of that day, one William Fiander, a Master Shipbuilder of some note. It was possibly, one of the repetitions of history that in the great struggle of our own time, one of the engineers called upon to give expert advice with regard to the use of reinforced concrete in the construction of ships as a special war measure should be none other than a direct descendant of the aforesaid William Fiander, namely, E. Fiander Etchells, the eldest son of Edward Ernest Fiander Etchells, of Romiley, near Marple.

Equally noteworthy is our President's professional lineage. Thus :—

Thomas Telford,
First President of the Institution of Civil Engineers (1820),
trained
J. Meadows Rendel, F.R.S.,
who trained
Sir Alexander M. Rendel,
who trained
Francis L. Lane,
who trained
Ernest Fiander Etchells.

TA
680
S75
V.12

Mr. Etchells was educated at the College of Technology, Manchester ; the Old Andersonian College, Glasgow ; and the University of London, King's College.

For several years he was Chief Assistant in the office of John Strain, Glasgow, Past Vice-President of the Institution of Civil Engineers.

He has been engaged on the design of Rolling Stock for the Pretoria and Pietermaritzburg Railway, Aerial Ropeways for Bolivia, Nitrate Plants for Chili, and was in charge of the Structural Steelwork for the reconstruction of the Lanarkshire Steelworks at Motherwell.

He was engaged on the Clyde Valley Electric Power Scheme and for the last twenty years he has been advising on the construction of buildings, structures and engineering plant, including cranes, cooling towers, foot bridges, pumping stations, electricity buildings, aeroplane hangars, balloon sheds, factories, warehouses, and engineering workshops and practically every class of building in which the advice of the engineer is sought.

In the prosecution of his profession as a Civil Engineer, he has ascended high factory chimneys, and descended coal mines to inspect the installation of pumping machinery, and he has travelled on the footplate of locomotives and tramped it on geological surveys.

He has also established a reputation as a lecturer by the lucidity of his explanations, and a search in the technical press would reveal many laudatory statements in respect of the interesting manner in which he imparts valuable information whatever the subject may be.

His rise to the Presidency of the Concrete Institute was no sudden incident in his career, but it was just a milestone along the pathway of a busy life.

He appears to enjoy an evening at a meeting or a conference better than most men enjoy an evening at the theatre. How else could one explain such a record as the following ?

1908. Elected as Member of the Concrete Institute.

1908. Appointed as Member of the Science Standing Committee.

1909. Elected Chairman of the Sub-Committee on Standard Notation.

1910. Elected as Member of the Council.

1911. Appointed Vice-Chairman of the Science Committee.

1912. Appointed as Chairman of the Improvements Committee.

1913. Appointed as Chairman of the Science Committee.

1913. Appointed as Member of the Board of Examiners.

1915. Appointed as a Member of the Parliamentary Standing Committee.

1915. Appointed Chairman of the Finance and General Purposes Committee.

1916. Elected as Vice-President.

1920. Elected as Chairman of the Literature Standing Committee.

1920. ELECTED AS PRESIDENT.

Our President, who is still a young man, as age is counted in the professions, has received wide recognition of his scientific and technical work as may be gathered from the following list of distinctions gained :—

Honorary Associate of the Royal Institute of British Architects.

Associate Member of the Institution of Civil Engineers.

Associate Member of the Institution of Mechanical Engineers.

Fellow of the Institute of Physics.

Member of the Mathematical Association.

Fellow of the Physical Society of London.

Member of the Science Standing Committee of the Royal Institute of British Architects.

Member of the British Engineering Standards Association.

Member of the British Joint Committee on Reinforced Concrete. (R.I.B.A.)

Membre du Conseil de la Société des Ingénieurs Civils de France.

Honorary Member of the Junior Institution of Engineers.

Statutory Qualification as District Surveyor (Diploma of the R.I.B.A.—1915).

Member of the Board of Examiners of the Concrete Institute.

Member of the Board of Examiners of the Society of Engineers.

Formerly Member of the Board of Examiners of the Institution of Municipal Engineers.

Member of the Committee of the Royal Institute of British Architects on Building Bye-Laws.

Formerly Member of the Research Committee of the Royal Institute of British Architects.

A National Silver Medallist of the City and Guilds of London Institute.

A First Prizeman of the Merchant Taylors Company.

It is not given to many to receive such recognition in the earlier years of their professional life, but if the past is any augury for the future, Mr. Etchells will be heard of many times again.

In conclusion it is permitted to us to express the hope that for many years to come his energy and his talents will be devoted to the service of his fellows.

A. E. Potter.

PRESIDENTIAL ADDRESS

By

E. FIANDER ETCHELLS,

A.M.Inst.C.E., A.M.I.Mech.E., Hon. A.R.I.B.A.,
Fellow of the Institute of Physics,
Member of the Mathematical Association,
Etc., Etc.

I have been too busy in these last few years to sit down and write addresses, but I have made a few notes, and the shorthand writers will take my remarks down verbatim. At one time I intended to give an address here on "Relativity," as I thought that would be a scientific problem of very great interest to the members, but the more I studied that question the more it appeared that it was a subject unsuitable for an address of one hour only. I fear that many of those popular articles which attempt, in a thousand words, to tell you all about "Relativity" fall very far short of their aim. I have also formed the opinion that two years is not too long to learn the responsible duties with which you have entrusted me, and I call upon all the distinguished gentlemen who have trodden the Presidential path before me to bear witness that the task which they undertook with the greatest reluctance was the preparation of a second Presidential Address before the same Institute. I make the suggestion that the future holders of this office should be absolved from this double penance. However, in the present instance my task is brightened by the pleasant privilege of formally introducing to the general body of members the new Secretary, Captain Maurice G. Kiddy.

THE NEW SECRETARY.

Captain Maurice G Kiddy was educated at Highgate School, where he gained many prizes. He is also something of a sportsman, for he has played cricket for the M.C.C. and for a minor county. He holds at least one unique sports medal—that of referee in the Dardanelles Football Cup Final! Captain Kiddy's business ambitions have always been towards secretarial and journalistic work, but war broke out, and he answered to a greater call—the call of an Empire loving peace, but unafraid of war. (Hear, hear.) Captain Kiddy enlisted in the London Rifle Brigade in August, 1914, and went with that regiment to France in November of the

same year, being one of the lucky possessors of the 1914 Star and Bar as a result. He was wounded in a hand-to-hand fight with the Prussian Guard just in time to be in London for Christmas, 1914. Upon recovery, he obtained a commission, and in September, 1915, he went with an infantry battalion to Gallipoli, and afterwards accompanied his regiment to Egypt. He subsequently returned to England, and was transferred to the Royal Flying Corps. He had varied experiences therein, and was promoted Captain, which rank he still retains on the reserve list of the Royal Air Force. He is a man who lets no time slip by unused, and the day after his demobilisation he entered the editorial offices of the London *Daily News*. About the same time he was responsible for a series of short stories in the *Royal* and other magazines. The stress and strain of war, and work, and the unceasing stress of daily paper journalism, proved too much for Captain Kiddy's health, and he resigned from the *Daily News*. Upon his recovery he was appointed Assistant-Secretary to the Society of Architects, with whom he remained for two and a-half years, until his recent appointment as Secretary of this Institute.

It is hoped that he may long be spared to us as a source of energy and a fountain of tact. Captain Kiddy was elected in 1919 a Member of the Institute of Journalists, and in November of this year he was successful in passing the tests and being elected a Fellow of the Incorporated Secretaries' Association, being, I think I am right in saying, the youngest Fellow on their lists. I hope he will live long enough to be their oldest. (Hear, hear.) As an organiser, Captain Kiddy has been, since the war, responsible for various public functions, and he recently managed the Royal Academy Ateliers Ball, with that effective skill which achieves results without noise or clamour. Gentlemen, I ask you to receive your Secretary with applause. (Loud applause.)

TRAINING THE CONCRETOR.

I have now to make a few suggestions for a scheme of systematic instruction in the theory and practice of concrete work. It is a remarkable fact that, although there has been an enormous development in concrete work during the last few years, and although it takes a prominent place in all forms of construction, so much so, that concrete may now be regarded as a distinct and individual industry, yet little provision has been made for systematic education in concrete theory and practice. What we want is a new type of mechanic or skilled workman: the expert concretor and block

setter. Up to now the education of the concretor has been left very largely to chance. Of course, technical Institutions include concrete as part of their engineering courses, but this is only incidental. We want to see more than this. Moreover, their courses of instruction have appealed rather more particularly to what is sometimes called "the directing class" rather than to the artisan class, on which, in the end, we must depend upon for good workmanship; the directing class being mainly responsible for good design. We want to see, not only good design, but good workmanship as well, and we want to see restored the old pride of workmanship, which, unfortunately, seems to have died down in certain quarters. Men are often not so proud of their calling as they ought to be, and, with a view to the future of the industry, it is imperative that provision be made for training the young handicraftsmen, and this, I think, might be done through the various trade schools and technical Institutes throughout the country. What is really necessary is that in the curriculum of these schools concrete should be regarded as an independent subject, and courses, both in simple theory and in good practice, should be arranged, since, in order to secure thorough efficiency and intelligent workmanship, the two must go hand in hand. The details of such a scheme would have to be worked out by a committee, but the outline, I imagine, would be something as follows:—In the first place, the syllabuses of technical schools are drawn up by the various county and local education authorities throughout the country, and have to be approved by the Board of Education. The first step, therefore, I think, would be to approach the Board of Education in order to ascertain whether the suggestion for such courses, if made, on the part of the institutions would be received with sympathy, because, if not, the whole thing would probably fall to the ground, since the various technical schools are largely dependent for their existence upon the Government grant. The next step, supposing the Board of Education agree, would be to approach the governing bodies of the various suitable trade schools with the suggestion that a short course of instruction, say six months each year—I do not think more would be necessary—in the theory and practice of concrete making and moulding, and block-setting, be included in the curriculum, such courses to be carried out under the Board of Education regulations for technical schools, etc., in order to qualify for the grant. Then I should suggest that a small committee of the Concrete Institute be appointed to draw up a suggestive course on modern lines.

which might be submitted to the authorities in question. This should include lectures on theory, the display of lantern slides and films, instruction and practice in manipulation, visits to concrete works, where students could see the work of concreting in actual operation, and to exhibitions of concrete products. Such a syllabus, which could be covered in a short course, should include:—

An outline of present-day methods of cement manufacture, with a visit to cement works;

The properties of Portland cement, with demonstrations;

Methods of testing cements and mortars;

The British standard specification;

The standard specifications of the Concrete Institute;

Fuller information as to the best methods for the storage of cement;

Information as to the wide application of concrete, with a visit to an exhibition of concrete products and the inspection of concrete work in progress;

Elementary study of aggregates and their suitability for various classes of work;

The importance of the cleanliness of aggregates, and how to secure it;

The importance of adopting the best methods, together with actual practice in grading, proportioning, mixing, gauging, placing, tamping, finishing and curing;

The various ways by which artistic surface effects may be produced;

Reinforcement—its function and advantages;

Simple tests for compressive and tensile strength; and

Block-setting.

One of the objects of these courses would be to produce skilled concretors and block-setters, who, by passing an examination and gaining a certificate, would have a definite status in the industry, which is already in existence, but which has yet to be organised. It is highly desirable that such organisation should occur, because those of you who have been on building estates have occasionally discovered that when concrete houses are being erected the bricklayers say this is their job, and they will go out on strike unless trade union men are employed. Now, there is much in connection with the erection of concrete houses which can be done by ex-Service men after a short period of intensive training. How can we get some agreement between two op-

posing ideals? The ideal of trade unionists is that all workers should belong to their trade union, or to some union. Now, it is desirable that all the ex-Service men who are willing to work should be given the opportunity to work. Would it not be advisable to have a block-setters' union, even though the men have not been trained as apprentices to bricklayers? I say let these men come into some block-setters' and concretors' union, and let them work side by side and in harmony with the bricklayers. We of this Institute can hardly put forward this suggestion, because, as a scientific body, we do not deal with questions of trade or politics, but your President is given the opportunity once a year to express his own personal opinions on such matters.

There is also another feature. It is hardly likely that the question of this concretors' union could be put forward from the employers' side, because, if that were done, certain advocates of the " class war " would imagine that they saw the " cloven hoof of the capitalists." They would say, " This is a scheme for the suppression of the workers; it is a scheme to reduce our salaries and lengthen our hours; the employers are trying to get an opposition camp to the bricklayers." Not at all. We want to find work for all who seek it, but all the same, the union could best be started by the ex-Service men themselves, because then they would see that it was entirely free from that terrible thing known as " the capitalist taint." Until harmony can be found between the trade unions, and until work can be found for the ex-soldiers, ex-sailors, ex-airmen, and all the other ex-Service men, I see very little hope of there being continued peace, prosperity and good trade in this country. It is one of the many essential things, and I throw this suggestion out for what it is worth, hoping that the artisans, or some of them, or even a few among them, will take up the case and found a trade union of their own. The employers do not object to union. It is disunion which creates mischief and distress. The best union, however, is a " Union between Capital and Labour, and Management." Each of these three groups can say to the other two, " United we stand, divided we fall."

EXAMINATIONS.

With regard to the examinations, these, I suggest, should be carried out under the auspices of the Concrete Institute, who already have the machinery for the conduct of these and more advanced examinations. Examinations might be held at centres, or in the institutions in which the instruction has

been given. In addition to the larger technical schools there is also a large number of evening schools and classes in which preparatory work might be done, and to which the scheme in a modified form might apply. The scheme we have outlined could not be carried out in its entirety at once, but, like other educational schemes, would have to be a matter of growth and development. We would thus be moving towards the organisation of the industry—the formation, so to speak, of a new craft which would in time have its own grades and develop on its own lines, as other trades have done. If such a scheme were carried into effect, safeguards for the trade unionists could be provided. and the foundation laid for thoroughly sound practice in an industry which will undoubtedly play a more prominent part in the future than it does even to-day.

STRENGTH OF BRICKWORK AND THICKNESS OF WALLS.

There has been a movement on foot gradually to reduce the thickness of walls, and I ask you not to be too impatient. I would remind you that in the Liber Alba, which was written some time in the Middle Ages, it was enacted that in London party walls should be 3 ft. thick. With the passing of time there has been a steady diminution in the statutory requirements of the thickness of party walls. In the London Building Act of 1894 there is a requirement of 13½ inches. In the regulations with respect to the construction of buildings wholly or partly of reinforced concrete, the minimum thickness of a party wall is cut down to 8 inches. Some of our friends here have expressed the opinion that a hollow brick with sides 1 inch in thickness is sufficient. I do not know whether these diminutions will continue, but there must be a limit. Perhaps someone will suggest that fire-resisting glazing should be sufficient; perhaps thin three-ply board may be recommended. Perhaps at some future Building Exhibition someone may come along with petrified tissue paper as the best substitute for brick party walls. It may be so, but I am not inclined to agree with them.

An examination of text books will reveal great divergencies as to the permissible loads on brickwork, and when appeal is made to the testing machine there are still disconcerting divergencies for which no adequate explanation appears to be forthcoming. There are, however, a few facts which force themselves upon our attention. One dominant fact is the necessity for making allowances for the psychology

of the bricklayer, the plumber and the electrician. After the loads of brick piers have been carefully calculated, it is disconcerting to notice the indifference with which a gentleman with a chisel will come along and cut a deep chase right across the most heavily loaded pier. Not infrequently, the chase is on the same side as that on which the load is applied. Indeed, in many cases the pier may be cut away, and a joist of inadequate strength may be thrown across the new opening. Thus, extra load is thrown on to the adjoining piers, which may, in their turn, be reduced in strength by the cutting of further chases. In addition to this, it is not unusual for crane makers to come along and propose to anchor heavy jib cranes on to brick pillars or piers of uncertain strength. With regard to the strength of brick pillars in themselves, one or two laws stand out from the weltering chaos of data. For example, tests show that:—

(a) The crushing strength of individual bricks may be 5 to 10 times the crushing strength of the pier as a whole;

(b) For any given quality of mortar, the strength of the pier varies with the strength of the individual bricks in the pier;

(c) For any given quality of bricks the strength of the pier varies with the strength of the mortar;

(d) If a mortar comprised of 1 volume of Portland cement and 4 volumes of sand be used in lieu of mortar comprised of one volume of freshly burned lime and two volumes of sand, the crushing resistance of the pier will be increased by an amount which may reach 25 tons per square foot. For example, a pier in lime mortar which might crush at 25 tons per square foot might be expected to resist 50 tons per square foot if cement mortar were used, as specified above;

(e) If blue bricks or other hard bricks are to be used, it is desirable that a rich cement mortar should be used therewith, in order that some advantage may be obtained from the hard bricks. On the other hand, if very soft bricks are used, the pier will be strengthened by the use of cement mortar instead of lime mortar;

(f) There is something beyond a simple crushing resistance involved in the failure of brickwork. There is, of course, tensile stress on the bulging side of

brick pier There is also a question of adhesion between the mortar and the brickwork. Many piers fail by splitting longitudinally, and the use of cement mortar is more efficacious in reducing this splitting tendency;

(g) Since lime mortar sets by the absorption of carbon-dioxide from the atmosphere, and since the outer coating of mortar sets first, and tends to protect the inner lime from atmospheric influences, it not infrequently happens that the lime mortar in the interior of a thick pier dries up rather than sets. An examination of the interior of brick piers which have failed appears to indicate that in some instances the mortar had never acted as a true cementing material;

(h) In order to increase the adhesion of the mortar to the bricks, it is essential that the bricks, if absorbent, should be well soaked before being used, and should be well drenched after being laid, otherwise there is a great risk of bricks absorbing the moisture from the mortar, and thus retarding or hindering the setting.

Some of these facts should be clear to designers, but nevertheless, the fact remains that an examination of piers which have failed discloses the fact that *somebody forgot*. Therefore, I do not apologise for reminding you of some known causes of failure. While still on the subject of bricks, one or two other points may be of interest to you. It is admitted that the strength of mortar, particularly lime mortar, is below the strength of the individual bricks. The cushioning effect of mortar, particularly in frogged bricks, is not always satisfactory. In some piers with frogged bricks the crushing resistance has been increased by laying the bricks on edge. I do not recommend this procedure, but mention it merely to show that with a given quality of mortar the strength may be affected by the method of laying and also to show that an inadequate cushioning effect is one of the sources of weakness of the frogged bricks in piers intended to carry heavy loads.

Careful tests also reveal the fact that brick piers undoubtedly display elastic properties. They shorten when loaded, and when unloaded they tend to resume their original length. As in steelwork, there is some strain which disappears on the removal of the load, and there is also some residual strain which cannot be eliminated. I hesitate to give any figure for the elastic modulus of a brick pier, be-

cause the data on this question is very scanty, and much remains to be done. There is an opportunity here for those university students who desire to undertake some original research.

With regard to eccentric loading, piers in the interior of buildings may have loads coming in on either side, but piers in the external walls may be subject to heavy eccentric loads. In the case of rectangular piers, the central load equivalent to any given eccentric load may be obtained from the formulæ:—

$$We = Wa(1+6.e/d)$$

where

We = *equivalent* central *weight* or load and

Wa = *actual weight* or load applied eccentrically

e = *eccentricity*, i.e., distance between the centre of the pillar and the centre of pressure of the applied load.

d = *diameter* of the pillar in the direction in which the load is applied, that is, the direction in which the eccentricity is measured.

The maximum intensity of pressure due to eccentricity of loading may be obtained from the equation:—

$$f = \frac{Wa}{A}\left(1+6\frac{e}{d}\right)$$

where

A = *Area* of cross section of the pier.

f = *flexural* stress at extreme "fibres."

The other symbols are used as previously. These equations are equations between *magnitudes*, and are, therefore, true for any system of units. For example, if Wa is in pounds, and A is in square inches, you will not need me to tell you what units f will be in. The value of e (*eccentricity*) should be taken from the centre of the pier to the centre of pressure of the applied load, but an important question at once arises: Where is the centre of pressure of the applied load? If the beam deflects, the load will bear on the front edge of the template or padstone. If the beam is very stiff, the pressure might be assumed to be nearer to the centroid of the area of the template. Whilst as a matter of fact, the centre of pressure must of necessity lie somewhere between the front edge and the centre of the template, it would not be unreasonable to take the centre of pressure as falling on the centre of the template in the case of very stiff girders, provided that a factor of safety was sufficient to cover the risks entailed.

THICKNESS OF WALLS.

In most areas of the Kingdom the thickness of walls is governed by the local Building Acts or Byelaws, but in cases where there is no such guidance the following formulæ may be used *faute de mieux*:—

Might I, however, be permitted to preface my remarks by saying that it is desirable to accept a brick having a nominal length of nine inches as a unit dimension in describing the thickness of walls.

Thus the term "one brick thick" means, not $4\frac{1}{2}$ inches thick, and not $8\frac{1}{2}$ inches, but 9 inches thick. If all our members were London architects, this explanation of the unit of thickness would be a waste of time, but in some districts the term "a one brick wall" would mean a wall $4\frac{1}{2}$ inches thick; and moreover some of our members may be chemists in Kamtschatka. Therefore prolixity may be less objectionable than obscurity or ambiguity.

The rule is simply this. The number of bricks in the thickness of the wall at the base shall not be less than

$$0{\cdot}5 + H/40 \text{ ft.},$$

where $H = Height$ of wall in feet and $n =$ the *number* of bricks as described above, expressed as an inequality the rule would be

$$n \not< 0{\cdot}5 + \mathrm{H}/40 \text{ ft.}$$

where the sign $\not<$ means "not less than."

The following table will give a few examples of the application of this rule, and a comparison of the results with the schedules of the London Building Act.

It is very curious that that rule may be applied to factory chimneys, domestic buildings or warehouses. The following further explanation is also desirable:—

(1) The rule assumes that in the case of domestic buildings and buildings of the warehouse class, the walls will be laterally stayed by floors in storeys not exceeding about 10 ft. in height;

(2) If the walls exceed 45 feet in length, another half brick should be added to the thickness given by the rule.

This rule is not to be used in lieu of calculations for the strength of the walls, but it is only intended as a general guide to their thickness. I believe architects of long practice

H = Height of Wall in Feet.	n = number of Bricks Thickness at the Base as required by Etchell's Formula.	London Building Act, 1894, Section 65. Factory Chimney Shafts.	London Building Act, 1894. Schedule for Domestic Buildings. (Short Walls.)	London Building Act, 1894. Schedule for Buildings of the Warehouse Class. (Short Walls.)
20	1	1	1	1½
20	1½	1½	1½	1½
40	1½	1½	1½	1½
41	2	2	2	2
61	2½	2½	2½	2½
80	2½	2½	2½	2½
81	3	3	3	3
100	3	3	3	3
101	3½	3½	3½	3½
120	3½	3½	3½	3½

will be amazed when they first discover a general law underlying what has been their practice in building or structures so diverse as country cottages and factory chimneys. The schedules to the London Building Act of 1894 were obtained from the Act of 1855 :—The schedules in the Act of 1855 were based on the schedules in the Act of 1844, amended as was thought proper at the time. But those who framed those schedules had no idea of a general formula. They were working on experience, of what had failed and what had stood. There was a lot of practical experience lying behind it all.

In rules developed over such a long period of time, by so many different architects, having such diverse ideas and without the guidance of a mathematical formula applicable to all three classes, it is only natural that inconsistencies and anomalies should have arisen Thus, in some English building Acts, in the case of domestic buildings, the change from one brick thickness to 1½ brick is made at 25 of height and not at 20 feet, as required by the rule.

Again, in the case of warehouse walls between 50 and 60 feet in height, certain building Acts require the walls at the base to be 2½ bricks thick instead of 2 as required by the rule.

Perhaps, when these Acts come to be amended, the Royal Institute of British Architects might desire to have the anomalies removed and the inconsistencies eliminated.

Indeed, if the wall thicknesses were regularised the schedules could be abolished entirely, and one or two easily-remembered rules could take their place.

BEARING PRESSURE.

As to pressure on founds and foundations, text books give the permissible *compressive stress* on brick piers having a height not exceeding 12 times the least diameter of the pier, and some such books do not permit any greater stress in the case of brick founds in which height is only a fraction of the width of the base, and in which the stressed area is surrounded by an unstressed area.

In such cases it would appear quite reasonable to allow a *bearing pressure* intensity which is 25 per cent. in excess of the average *compressive stress* on the slender pier referred to above. It will be noticed that I am using the term " bearing pressure " as meaning a compressive pressure applied on a central area which is surrounded by an unloaded area. I am not forgetting that in many instances the size of the brick founds is determined more by the bearing resistance of the earth below than by the pressure superimposed upon the brick founds.

FOUNDS AND FOUNDATIONS.

In passing I would like to say that I personally approve of the distinction made by one of our members, I think it was Mr. Phillips, who suggests that having regard to the antiquity of the term "foundations," it would be useful to refer to the concrete or brickwork as being the founds, and to the natural earth or rock under the "founds" as being the "foundations." I am, however, not committed to any particular term if any pair of terms can be found to distinguish between the "founds" and the "foundations." To those who think there is no necessity for any discrimination, I would ask what is the pressure on the foundations? Is it the pressure on the truncated pyramid of brickwork below the steel stanchions, or is it the pressure on a limited area of the natural ground? If it is the pressure on the natural ground, what does "putting in the foundations" mean?

CONCRETE COTTAGES.

From time to time we are treated with dissertations on something alleged to be absolutely novel in the concrete cottages. The inventor explains to us that there never has been such a system as his, and the world has been waiting through many ages for his advent. He speaks as though his wall were the cheapest ever. It is stronger than anything the engineers could devise, and more beautiful than anything the architects could conceive. As many inventors have the same opinion, they cannot all be true, and even the newspapers are misled. In last Friday's *Times* there is a very nice little paragraph about some concrete houses, and a nice little picture, and the man explains that his scheme is just it, that he invented it himself, and he came from America. (Laughter.) That man did not appear to know that the dovetail ties, which he appraised as a novelty were invented in the adjoining Borough of Lambeth in 1878. Also, with regard to his pouring concrete into the hollows in walls that was not the new thing he thought. Vitruvious describes it as something well established. So we need not talk of the novelty. Then, again, in London in 1872 there was a Mr. Tall, who was inventing walls. He said, "The object of his invention and patent was to effect an economy in building dwelling houses, cottages, garden walls and other erections by forming such structures in concrete combined with brick rubbish or other hard, and durable substance, in place of employing costly brickwork or masonry." The process for effecting

such construction was by the use of a certain kind of moulding or framework affixed from stage to stage as every 18 inches of material is raised on the walls, and it becomes set and hardened into a solid mass. That 18 inches is very curious, for it is a most common figure for shuttering nowadays in the patent systems that are recommended as effecting such economies. Essentially, this framework is what is patented, as concrete of all kinds is a well-known and an ancient compound beyond the reach of Patent Law.

About fifty years ago there were also the patents of Mr. Phillip Brannon. People who suddenly get brain-waves about new forms of construction would do well to turn up the oldest files in the Patent Office, take a copy home, and read it at their leisure, then humble themselves and not think that the inventive arts began when they sat down to meditate. Mr. Brannon's mode of constructing concrete buildings is claimed as applicable to all parts of buildings "upon the principle of monolithic construction of cement concrete, or concreted earthy or other materials in combination with metallic, fibrous or lamellated substances, with a view to rendering buildings more durable, fireproof and healthy than those of the present construction, and consists in employing for the above purposes a sustaining metallic skeleton or framework, firmly bolted or bound together, upon which is stretched open wirework, the whole so arranged as to admit of being entirely or partially enclosed or embedded in the said concreted or other materials composing the structure or portions thereof, and thus secure perfect bonding of the same into a solid and coherent mass; and to further increase the coherency and solidity of the mass, and render the same capable of resisting transverse and tensile strain when applied to floors, roofs, arches, bridges, viaducts and aqueducts, the patentee intimately commingles with the concreted materials upon the principle which he terms stone-felting or litho-felting process, and interposes between successive layers thereof or between courses or layers of bricks, tiles or stones, or any lamellated material, strands or fabrics or wire or fibrous substances . . . . to effect the bonding of the mass." The process is thus fully described in the patent specification.

INSPECTION AND SUPERVISION.

One important matter recommended fifty years ago was the appointment of a Clerk of Works or other foreman, *subject to the approval of the District Surveyor*, in addition

to the usual supervision by the District Surveyor. As the latter could not always be present, it was hoped that such a person as a Clerk of Works, acting under the condition prescribed, would have a beneficial effect in keeping such works up to a satisfactory standard.

If such inspection was desirable then, what of its necessity under the difficult and complex conditions obtaining to-day in extensive reinforced concrete works.

FIRE RESISTANCE OF WOOD.

During the past year the question of the fire resistance of wood has been made a subject of further investigation, and the results of some of the experiments may be of interest to our members. The first impression gained from the examination of tests of wood is the striking diversity of the results. At first it appears impossible to lay down any general rule which can be applied with confidence to particular cases, but gradually, certain leading principles begin to emerge from the disordered array of chaotic figures. One of the points to be borne in mind is that in addition to the laws of chemistry and the laws of physics, we must recognise the operation of the laws of biology. Trees are living things. They are individuals, and each tree has an individual history. It has, or has had, parents and grandparents, and all the more remote ancestors. The individuality of trees is well recognised in some verses, I believe by Heine:—

> "A pine tree standeth lonely,
> On a north and upland plain,
> It standeth whitely shrouded,
> With sorrow, and sleepeth there."

> "It dreameth of a palm tree,
> Which far in the East alone,
> In solemn silence standeth,
> On its ridge of burning stone."

That piece of oak is split! Was it due to shrinkage of the wood when it was being dried in the hot air chamber last week, or was it due to the excessive stress in some great hurricane which nearly uprooted the sapling oak two hundred years ago? "Twist the green bough of the sapling, and the gnarled and knotted oak will tell of thee in centuries to come."

In the case of the chemical elements, they are for all practical purposes exactly similar, however they may have been produced, but in the case of persons, our characters are influenced by our heredity and our environment. Thus

it is with trees. The quality of the wood depends upon heredity and environment of the tree. In speaking of the chemical elements I said that they were for all practical purposes exactly similar. This reservation was added in order to allow for a suggestion which has been made from time to time and which was repeated at the meeting of the British Association during the current year, to the effect that possibly the fractional figures in the atomic weights of the elements may indicate an admixture of atoms of the like kinds, but having different weights or different quantities of atomic energy. I was also making allowance for the suggestions of the students of relativity, who hold that the mass of an atom may vary with its velocity. However, reverting to the ques tion of timber, I would remind you that the strength of timber depends *inter alia* upon its age, and a tree contains parts having many different ages. One cannot give a pre-cise answer to the question as to how old is *that* piece of wood, because, as a matter of fact, the rings which are nearer to the heart are older than the rings nearer to the bark. In the extreme case of a living oak, the heart wood might be 200 years old, while the outermost ring of sap wood might be no older than one year. Bearing in mind that the strength of a fibre of wood depends *inter alia* upon its age, and re-membering that if we pass from the pith to the heart wood, and from the heart wood to the sap wood, and then again to that active living layer or cambium, and then to the bark, we should find that each ring of wood was younger than its neighbour " on the inner circle." How, then, can we expect these different parts to have the same physical strength and the same resistance to fire? Neglect for the moment the irregularities due to the multiplicity of the branches and to the accidents of circumstances of a particular tree, we notice that the annual addition to a tree's growth approximates in form to a conical sheath tapering to a point at the topmost bud of a tree. This conical sheath of living tissue is the source of the new wood. " It is in process of becoming " wood. Its quality will depend, among other things, upon the amount of sunshine and rain in the particular locality in which it lives as cambium. How, then, can it be expected that the strength of one fibre of wood can be exactly equal to the strength of another fibre of wood formed by the same tree, but under different conditions of weather?

There are many systems or classification of trees, and none of them are entirely satisfactory, but each of them is suitable for some particular purpose. The botanist may divide trees into two groups :—

(a) Those which grow by the addition of external layers, and are known as exogenous;
(b) Those in which the growth is internal, known as endogenous trees.

Exogenous trees may be divided into
(a) Broad-leafed trees, such as the oak and maple, and
(b) Narrow-leafed trees, such as the fir and the lime.

The carpenter may divide woods into two kinds, hard woods and soft woods, but these classifications may run counter to some of the other systems. For instance, broad-leafed trees may produce hard woods, and narrow-leafed trees may produce soft woods, but the poplar is a broad-leafed tree, and yet it produces soft wood. What we are leading up to is how can we tell whether a certain wood can be fire-resisting. Is there any classification to help us? There is another common system of classification: Woods may be classified as (a) conifers and (b) non-conifers.

Firemen and insurance engineers may divide woods into two categories, (a) fire-resisting and (b) non-fire-resisting. Conifers such as the Scotch Fir and Pitch Pine are not fire-resisting; non-conifers, such as the oak and teak, are fire-resisting, but it cannot be said that the conifers are not fire-resisting and non-conifers are fire-resisting, because examples in each category may be found, and these examples would prove the exception to any rule which might be formulated.

It cannot even be said that the presence of resin or gum in a tree would indicate that the wood was not fire-resisting, because jarrah expels drops of resin and gum when burned; yet, nevertheless, the wood burns indifferently; embers soon die out, and leave carbonised wood. Despite its resinous contents, it is, nevertheless, one of the least inflammable woods.

Wood in itself consists almost entirely of cellulose. The cells, however, may contain various amounts of resin, gum, turpentine or other essential oils. The great difference in the specific gravity in different woods is due to differences of structure rather than to differences of chemical constituents. If wood fibre be broken up, the specific gravity is about 1·5, whether the wood be a heavy hard or a light soft wood. It is indeed surprising that the fibres have about the same specific gravity. It means that the light woods are filled with air spaces, which are lost when the fibre is broken up for the purpose of testing the specific gravity. Wood floats on account of the cells being filled with air. If the wood is dense, like certain specimens of oak I have handled, or if it

remains so long in the water that all the cells have become saturated, the wood would sink.

It would not be fair to compare a resinous wood with a non-resinous wood, but if two specimens of the same kind of wood were taken, one more dense than the other, the denser wood would have the greater fire resistance, and for two reasons:—

(1) There is more wood to burn through;

(2) The specimen contains less air, that great supporter of combustion.

If the cells were filled with some inflammable material it might be a better fire-resistant than woods containing many air cavities. One dare not draw general rules from the exceptional specimens; nevertheless, although generally oak is a better fire-resistant than pine, it is possible to find examples in which a dense and voidless fir may be a better fire-resistant than a light and cellular oak. But despite the old proverb, the exception does *not* prove the rule.

Woods can only be arranged according to their fire-resistant properties after the result of exhaustive tests by flame and fire. Recently four investigators have been each separately comparing the fire-resistance of woods, and they were taking the oak on the one hand, and pine on the other, as typical specimens, and each of them found specimens of pine which would resist the fire better than certain specimens of the oak. Each of them thought he had made a mistake, and said nothing about it, but incidentally, in conversation, each mentioned the fact that he had found that to be the case, and in each case the others admitted "so did I." While in general oak is the better fire-resistant, yet you can find specimens of pine here and there which do resist fire better than oak. But you must not draw a rule from that, because you can say that as a rule men are stronger than women, but that does not say that there may not be some women stronger than men.

Inflammability may indicate a high resinous content, or it may indicate a highly cellular structure. Advantages and disadvantages may cancel each other or accentuate each other. Among woods generally recognised as being generally fire-resistant one should include oak, jarrah, teak, karri, walnut and mahogany. The best testing is by fire, but as this is not always practicable, the next best test is by ascertaining the specific gravity of the wood.

At the risk of wearying you, I will venture into further detail. If the specific gravity of various kinds of wood were

to be plotted as the abscissa on a diagram, and if the flexural strength of the wood were to be plotted as the corresponding ordinates, the resulting diagram would have the appearance of the conventional representation of a very zig-zagged streak of lightning. Nevertheless, despite the inevitable exceptions, there is a definite indication that, on the whole,

(a) The heavier kinds of wood are stronger than the lighter kinds of wood.

(b) For any given kind of wood, the heavier specimens are stronger than the lighter specimens.

Both these rules would appear to be the natural consequence of the fact that the weight of a given specimen of wood depends upon the closeness of the fibres, and is not due to any differences of chemical constitution.

In brief, the heavier specimen is stronger because it contains *more wood*. The next fact in the chain of argument is the statement that for beams of a given size the load should be proportionate to the flexural strength. Now, if we have a series of cantilevers all the same size, and with no load except that due to their own weight, then the flexural stresses will, automatically, be perfectly and absolutely proportionate to the weight of the specimen.

Therefore to test the fire-resistance of beams, at flexural stresses in exact proportion to the strength of the wood in question, all that is necessary is to obtain cantilevers of a standard size, not subject to any extraneous load, and place a standard flame at a standard distance from the extremity. In one of the series of tests conducted by the writer the specimens were one inch by one inch in cross section, and the flame was applied 10 inches from the extremity. An ordinary Bunsen burner with a flame at a standard height was used to ignite the specimens. The flame was arranged at a height which caused the flame to embrace both flanks of the specimen.

About ten specimens of each kind of wood should be tested. The average time required to bring about the collapse of the cantilevers should be noted.

The relative fire-resistance is therefore measured in terms of the period of time during which the cantilever resists collapse.

STANDARD NOTATION.

With regard to standard notation, the principal improvement during the year is one which should appeal particularly to editors and publishers. It is an improvement which considerably cheapens the cost of printing engineering formulæ. It is nothing less than the abolition of subscript

letters (that is, suffixes or inferiors) which were introduced by Leibnitz (1646-1716). This improvement, coupled with the more general use of the solidus or sloping dash, renders it possible to set engineering formulæ up on the typewriter or on the linotype or monotype or other composing machines, whereas formerly it was necessary to employ handwriting instead of typewriting, and hand-setting instead of machine-setting. The elimination of the subscript letters has been rendered possible by the use of

(1) The *high* point which is used as the decimal separatrix—for example, $7{\cdot}5 =$ seven and five-tenths;

(2) The *middle* point, which is used as a sign of multiplication between two letters which represent the two factors—for example, $a \cdot b = a$ multiplied by b;

(3) The *low* point, which is used as a factorial separatrix instead of brackets, between groups of factors. For example $6.e/d$ means $6.(e/d)$. This low point is only used for clarity in cases where its omission would not affect the correctness of the numerical result.

Fuller details of these improvements, together with a Standard Notation of the Concrete Institute for calculations in respect of reinforced concrete and other branches of structural engineering, including the symbols approved by the Concrete Institute on 14th April, 1920, will be found printed in a "Treatise on Reinforced Concrete," by W. Noble Twelvetrees. The Standard Notation of this Institute will also be found in use in two forthcoming books, both by members of this Institute, namely, Dr. Faber, who is publishing a book entitled, "Reinforced Concrete Simply Explained," and in another book by Captain Harrington Hudson entitled, "Reinforced Concrete: A Practical Handbook," to be published shortly by Messrs. Chapman & Hall.

RECENT DEVELOPMENTS IN THE THEORY OF REINFORCED CONCRETE.

A few words, now, with regard to the report of the Royal Institute of British Architects on reinforced concrete, and also a few words with regard to the trend of expert opinion in respect of the evolution of the theory of reinforced concrete. In the ten years which have elapsed since the R.I.B.A. published their second report, much additional data has been obtained. It might be useful if I were to give one or two examples of the trend of opinion among the experts.

Ten years ago, in the endeavour to seek generality in the pillar formulæ, and to devise a rule which would enable Mouchel and Considere pillars to be brought to the same standard of comparison, it was recommended that only the concrete within the hooped core, or lateral binding, should be considered as load bearing material. It was also considered that the concrete outside the binding would flake off long before the ultimate load is reached.

At a later stage, it began to be realised that a large percentage of pillars were eccentrically loaded and that the pillars were, in consequence, liable to flexure, and the concrete outside the binding would, in some cases, undoubtedly be in compression. Some of these facts were foreseen at a very early stage, but as there was at that time prejudice in some directions against any calculations of the effect of eccentricity, it was felt that the difficulty could be partly met by allowing only the area of the core to be considered as constituting the compression area of the pillar, and to allow the concrete outside the laterals on one side of the pillar to be useful in resisting the flexural stresses due to the eccentricity of the load.

At a later stage, when the prejudice against calculations for eccentricity of loading had somewhat subsided, it was recognised that it would be unreasonable not to allow all the concrete in the pillar to be considered as a part of the pillar, particularly if the question of flexural stresses was under consideration. It was felt that for the time being, at any rate, the member was in effect a vertical beam subject to direct compression in addition to the flexural stresses.

Going back to an earlier stage in the evolution of reinforced concrete, it will be found that beams were frequently designed as having ends freely supported by a series of pillars in line. Naturally enough, cracks were found in the tops of the beams over the supports. To remedy this defect, an arbitrary amount of steel was inserted over the supports, but the amount was not calculated on any scientific basis.

At a still later stage, beams were designed as being purely continuous, and were designed to resist variations in the incidence of loading so that no cracks would occur whether all the bays were loaded or whether any of the bays were unloaded.

In the meantime, two earnest students of reinforced concrete (1) Dr. Faber in England and (2) Mr. Harrington Hudson in India, were engaged on the laborious calculations necessary to find the general formulæ which would take into account the bending moments on a series of pillars rigidly

connected with the beam, constituting one truly monolithic structure subject to elastic deformations.

The result of Dr. Faber's investigations were published first, but Mr. Harrington Hudson's work, which was published later, was quite independent.

Dr. Faber obtains his results by considering the slope of the elastic line at the ends of the beam. His calculation involves the use of a stiffness factor which is the heterogeneous rate subsisting between the inertia moment and the length of the member.

Mr. Hudson obtains his results by an ingenious adaptation of the theorem of three moments, and he uses two pure ratios which are obtained, in one case, by finding the pure ratio between the inertia moment of the beam and the inertia moment of the pillar, and in the second case, by finding the pure ratio between the length of the beam and the height of the pillar.

Dr. Faber introduces a constant depending on the condition of the other end of the pillar, i.e., depending upon whether the other end of the pillar is free, fixed, or constrained to take up a slope of the same magnitude and sign, or equal in magnitude, but opposite in sign to the slope at the connection between beam and pillar.

Mr. Hudson assumes that the base of the pillar, when constructed on a monolithic footing, is perfectly fixed. He then locates the points of contra-flexure in the pillars by means of a cubic equation. In order to make his formulæ more general, he uses a qualifier denoting a wide range of different distributions of weight on the beam and a ratio of weights denoting the relation of dead and super-imposed weight on the spans of a beam.

To facilitate the use of their respective systems in practice, Dr. Faber plots his results on curves, whereas Mr. Hudson prefers to use a set of alignment charts and a diagram.

There is no doubt that all those who have to formulate rules for monolithic construction must hereafter take into account the work of these two engineers, who, I am proud to say, are both members of the Concrete Institute.

It is, however, possible that for some time to come there will be engineers who would prefer to design their structures on the assumption that all loads on pillars are axial, and all beams are freely supported. Then afterwards, recognising that their assumptions are not in accordance with fact, they will add an arbitary amount of steel to minimise the tendency to fracture.

CONCLUSION.

Now just one word, before I close, with regard to the question of relativity. I was going to give an address on that alone, but, as I say, it was found that the subject was of such complexity that it would be impossible, in even the longest evening, to make you acquainted with all the definitions of the new concepts involved; but if we may leave the equations aside for one moment, we may say that the principle of relativity is perfectly familiar to the philosophers, but what I press most is that scientists are waking up to the fact that many of their concepts are only working notions to enable them to predict results, and the laws of science are becoming recognised as useful rules and not of necessity, the absolute unqualified truth irrespective of their setting. Now some part of this idea was familiar to someone who lived in the adjoining borough of Chelsea, many years ago—Carlyle. He had looked at some of the systems of nature in his days, and found contradictions amongst them and in speaking of them, he says:—" Systems of Nature! To the wisest man, wide as is his vision, Nature remains of quite infinite depth, of quite infinite expansion; and all experience thereof limits itself to some few computed centuries and measured square miles. The course of Nature's phases, on this our little fraction of a Planet, is partially known to us; but who knows what deeper courses these depend on; what infinitely larger cycle (of causes) our little epicycle revolves on? To the minnow, every cranny and pebble, and quality and accident, of its little native creek, may have become familiar; but does the minnow understand the ocean tides and periodic currents, the trade winds and monsoons, and moon's eclipses; by all which the condition of its little creek is regulated, and may, from time to time (unmiraculously enough) be quite overset and reversed? Such a minnow is man; his creek this planet earth; his ocean the immeasurable all; his monsoons and periodic currents, the mysterious course of Nature " through countless ages of limitless Time.

Ladies and gentlemen, my task, nay my pleasure, of addressing you must finish with this thought, that the most enduring vestige of any civilisation will probably be some buried and forgotten foundation bed of concrete. Steel will rust, wood will perish, bricks may crumble and decay; but concrete foundations buried deep in the earth will endure unto the End.

Mr. F. E. Wentworth Shields, in proposing a hearty vote of thanks to the President for his address, said he could appreciate the President's remark that the preparation of a second Presidential Address on re-election to the Chair was no light task, and that the custom might be abolished. He believed it had been abolished, on one occasion, at all events, but he was sure that in this case they all agreed that if Mr. Etchells were elected fifty times he would with facility give them fifty Addresses, and each one would be better than the last. He had that charming gift of making every subject that he touched so interesting and so lucid that one could listen to him again and again with renewed pleasure. Whether he was quoting poetry or telling them about the ancient Egyptians, or the sins of the modern contractor, he delighted them always, and they would listen to him as often as he liked to speak to them. Mr. Etchells has, as they all knew, been one of the hardest working Presidents, if not the hardest working President that they had had. Every member of the Institute knew a good deal about the excellent work he had done. His work on standard notation, which he had quoted towards the end of his Address, was a lasting honour to the Concrete Institute, and especially to its President. Those who had been associated with him knew the number of hours and the hard and good work he had done to bring the Institute to be the most helpful engineering society in the profession. He had spoken towards the end of his Address of the wonderful stability of concrete, but he (Mr. Wentworth Shields) believed that so long as the Institute had such Presidents as Mr. Etchells, there would be something even more stable than concrete, i.e., the Concrete Institute.

Mr. H. J. Deane seconded the proposal. His own experience of Mr. Etchells had been confined to the last few years, but he very heartily endorsed what Mr. Wentworth Shields had said as regards his duties, and his tact and ability in dealing with some of the most difficult and delicate matters that had had to be considered.

Mr. W. J. H. Leverton said he had been asked to say a few words in support of the vote of thanks, but he was afraid that to do so would be what the theologians called a work of super-rogation. He was very pleased to hear Mr. Etchells refer to the boom in concrete building which took place some 40 years ago. He referred to Tall and Brannon; who built a number of houses at Enfield. There was also Drake, who built houses at Wandsworth Common, and there was also Bloomfield, so that concrete was not such a

young thing as some imagined. Most of those buildings were plain concrete and not reinforced. With regard to Mr. Etchell's charming Address. he was always so varied and entertaining that it was delightful to listen to him whatever his subject.

The President, acknowledging the vote of thanks, said that his work was his hobby, and his hobby his work. He was doing, and intended to do, whatever he could to make every Session of the Institute better than the one that preceded it. And he closed his Address with an assurance that his efforts to stabilise and increase the status of the Intitute would not cease with his tenure of office.

PROCEEDINGS.

The 106th ordinary general meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, November 24, 1921, at 7.30 p.m. The chair was taken by the President, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the previous minutes were taken as read.

The following members were elected in accordance with Article 8 of the Articles of Association :—

Members.

Atkinson, George Walter, F.S.Arc., 1, Mark Lane, Leeds. Beswick, William, A.R.I.B.A., 19, Newgate Street, Chester. Bindley, Herbert Duncombe, A.M.Inst.C.E., Office of E.W.W., F.M.S.R., Kuala Lumpur, F.M.S. Bowden, James Albert, F.S.Arc., 31, Craven Street, Strand, W.C.2. Brazier, Clifford Claude Hendy, A.M.I.Mech.E., c/o White's South African Cement Company, Ld., White's Station, near Nentersburg Road, Orange Free State. Brewster, John Thomas Mervyn, Pongaroa, New Zealand. Browne, Ernest, A.M.S.E., Borough Engineer's Office, Croydon. Burnett Brown, Alexander, F.S.Arc., F.S.I., Lennox House, Norfolk Street, Strand, W.C.2. Campion, Frederic Eugene, A.M.Inst.C.E., District Engineer's Office, L.B.S.C.R., E. Croydon. Clark, John Percy, F.S.Arc., A.R.C.A., Regent Chambers, Wednesbury. Corkhill, Thomas, 70, Howard Road, E.17. Cotterell, Ernest Lawrence, M.S.A. 57, Raleigh Road, Wolverhampton. Dickinson, Alfred William, Caxton House, Westminster, S.W.1 Doi, Matsuichi (Member of Inst., of Japanese Architects), c/o Okwa & Co., 53, New Broad Street, E.C., and College of Technology, Tokio, Japan. England, John, A.M.Inst.E. (Australia), Commerce Buildings, Ash Street, Sydney, Australia. Fearn, Stanley Walter, F.N.Z.I.A. 156, Upper Willis Street, Wellington, N.Z. Foster William Frederick, F.S.Arc., F.S.I., M.R.San.I., 6, Great Russell Street, W.C.1. Henry, Thomas William, F.S.Arc. (*P P. Ulster Society of Architects*), Ocean Buildings, Donegall Square E., Belfast. Hider,

ERNEST JAMES WEDLOCK, F.I.A., M.S.A., 11, Avenue Chambers, Southampton Row, W.C.1. HOPKINS, PERCY ALFRED, F.I.A., M.S.A., M.R.San.I., 25, Mortimer Street, W.1.; INGLIS, MAJOR THOMAS STEWART, D.S.O., F.S.Arc., M.I.A., 15, Copley Park, Streatham, S.W.16. KESTEVEN, LEOFRIC, M.S.A., M.R.San.I., P.W.D., Kuala Lampur. F.M.S. LAWRENCE, RONALD MACKENZIE, A.M.Inst.C.E., c/o F. Sage & Co., Ld., Casilla Correo 1699, Buenos Aires. LEWIS, JOHN STUART, 4, Central Buildings, Westminster, S.W.1. MAWSON, EDWARD PRENTICE. F.S.Arc., High Street House, Lancaster. MONRO, WILLIAM ERNEST, F.S.Arc., Lic. R.I.B.A., 46, Lincoln's Inn Fields, W.C.2. MONSON, MAJOR EDWARD CHARLES PHILIP, T.D., F.R.I.B.A., F.S.Arc., F.R.San.I. F.I.S.E., F.I.A. (*Past President, Society of Architects and Institution of Sanitary Engineers*). Finsbury Pavement House, E.C.2. MORRIS, GEORGE THOMAS. 37, Chancery Lane, W.C.2. OSBORNE, SAMUEL, c/o Messrs. Chas. Heathcote & Sons, Lloyds Bank Buildings, King Street, Manchester. PARKIN, WILLIAM GORDON, A.R.I.B.A., c/o Hemmings & Berkley, Hankow, China. PHILLIPS, ERNEST WILLIAM, City Architects' Department, Town Hall, Manchester. ROBINSON, FRANK H. G., 207, Northumberland Park, Tottenham, N.17. ROSE, HAROLD GREENWELL, M.B.E., A.M.Inst.C.E., 39, West Sunniside, Sunderland. SADGROVE, EDWIN JAMES, F.R.I.B.A., F.S.Arc., F.I.A. (*Past President, Society of Architects V.P., Inst. Arbitrators, 1921)*, 9a, Little James Street, Bedford Row, W.C.1. SHOEBOTTOM, WILLIAM HENRY, A.M.I.A.E., Imber Court, East Molesey. SMART, HENRY CHARLES, F.S.Arc., 73a, Queen Victoria Street, E.C.4. VICKERY, THOMAS SIDNEY, F.S.Arc., 50, Gresham Street, E.C.2. WALLIS, THOMAS, F.S.Arc. (*V.P., Society of Architects, 1921*), 29, Roland Gardens, S. Kensington, S.W.7. WATSON, WILLIAM HAROLD, F.S.Arc., Lic. R.I.B.A., Barstow Square, Wakefield. WILLIAMS, RICHARD JOHN, F.R.I.B.A., M.S.A., Parkstile Chambers, Kettering.

Associates.

MCLEOD, WILLIAM, M.A.C.I., c/o Bank of New Zealand, 1, Queen Victoria Street, E.C.4.

Associate Members.

BRIDGES, GEORGE PERCIVAL, Industrial Constructions, Ld., 44, Grosvenor Place, S.W.1. BRUNE, HERMAN, P.W.D., Union Buildings, Pretoria, S.A., BUTLER, HORACE GEORGE ST. PAUL, 42, Lancaster Gate, W.2. CARTLEDGE, HERBERT HENRY, A.M.Inst.E. (Australia), Furline Street, West Adelaide, South Australia. COPE, GEORGE IVAN, M.C., B.A., 15, Western Parade, Southsea. CORKILL, FRANCIS MALCOLM, A.M.Inst.C.E., B.E., M.Sc., P.W.D., Kuala Lumpur, F.M.S. CROUCH, GERALD LINFORD, P.A.S.I., 3, Foxley Road, Nutfield Road, Thornton Heath, Surrey. DE NORONHA, DON MARCOS DA SILVA, 36, Devonshire Place, W.1. ELLIS, JOHN, A.M.Inst.C.E., 9, Belvedere Road, Wallasey, Cheshire. FARROW, ALBERT VICTOR, A.M.S.E., 166, Hornby Road, Bombay. FOX, CHARLES HORACE, O.B.E., B.Sc., F.S.I., 7, Rawson Street, Halifax. HODGE, WILLIAM JAMES, 9, Langworthy Road South, Weaste, Manchester. KNUTSON, LEIF, P.O. Box 4843, Johannesburg, S.A., and P.O. Box 1711, Durban, Natal, S.A. LOW, VICTOR THOMAS, 4, Handel Street, W.C.1. LOWE, CLARENCE VAN RIET, B.Sc., P.W.D.,

Union Buildings, Pretoria, S.A. MERMAGEN, HERBERT FRANCIS GARFIELD, "Inverclyde," Holland Road, Cheltenham. MILLER, JOHN CLYDE, 162, Gatehouse Street, Parkville, Melbourne, Australia. MULVANEY, JOHN LEE, c/o McAndrew, Forage & Mulvaney, Basrah, Mesopotamia. MURGATROYD, NORMAN BRINTON, 148, Smedley Road, Cheetham, Manchester. SCHOFIELD, REGINALD WILLIAM, A.M.S.E., 16, Alfriston Road, West Side, Clapham Common, S.W.11. STALKER, ALBERT WILLIAM, A.M.Inst.C.E., c/o S.C.E., H.M. Dockyard, Portsmouth. TAYLOR, WILLIAM JOHN, A.R.I.B.A., 52, High Street, Inverness. TELLERY, FRANK IGNATZ, A.R.I.B.A., M.S.A., c/o Alliance Bank of Simla, Ld., Calcutta, India. WETHERBY-WILLIAMS, A. V., 37, Parliament Street, S.W.1.

Licentiates.

WILD, WILLIAM, 18, Cort Street, Blackhill, Co. Durham.

The Secretary announced the election by the Council of the following Graduates and Students :—

Graduates.

JOYCE, HAROLD FREDERICK WILLIAM, 11, Ethelden Road, Shepherd's Bush, W.12. NAYLOR, ALLAN, 47, Lister Avenue, Bowling, Bradford. NILLSON, DOUGLAS, 183, West Regent Street, Glasgow. NIXON, JOSEPH, 103, John Street, Workington. RADNALL, HERBERT, 4, Brookhill Road, Ward End, Birmingham.

Students.

DODSON, L. W., 27, Rushden Road, Manor Estate, Levenshulme, Manchester. DUNN, THOMAS, JUN., 170, Dunbaston Road, Partick, Glasgow. NAUIDA, M. GOSPALAK VISHNU, Central Provinces, P.W.D. Chanda, India.

The President then delivered the Presidential address at the commencement of his second year of office, and a vote of thanks to the President was moved by Mr. F. E. WENTWORTH-SHIELDS (Past President), supported by Mr. H. J. DEANE and Mr. W. J. H. LEVERTON (Members of Council), and was carried with acclamation.

The proceedings then terminated.

A CONCRETE INSTITUTE MASONIC LODGE.

Some members of the Concrete Institute are desirous of founding a Masonic Lodge, associated with the Institute. Members of the Institute interested are invited to communicate with the Secretary of the Institute, Captain M. G. Kiddy, 296, Vauxhall Bridge Road, S.W.1.

THE CONCRETE INSTITUTE YEAR BOOK.

In place of the former issue of a periodical " List of Members " (last issued in 1919) the Council has decided to issue a Year Book to contain, besides the list of members, matter of utility and value to members of the Institute.

The first issue will be published on April 10, 1922, and in the meantime members are asked to communicate any changes of title, address, &c., to the Secretary, so that the next list may be as accurate as possible.

THE LIBRARY.

The Secretary will be glad to receive donations of books, catalogues, tables, and section books dealing with concrete, reinforced concrete, structural steel, engineering, &c., to place in the library.

Authors and publishers presenting books may have them reviewed in the " Journal " from time to time as space permits, if they so desire it.

THE ANNUAL DINNER.

Members who have not already done so are asked to make early application for tickets for the Annual Dinner which is being held at the Savoy Hotel, London, W.C.2, on Thursday, February 2, 1922, at 7 p.m., for 7.30 p.m.

The price of single tickets is 15s., and of double tickets 27s. 6d. (not including wines), and application should be made to the Secretary of the Institute, 296, Vauxhall Bridge Road, S.W.1.

FINANCE AND GENERAL PURPOSES COMMITTEE.

The new Chairman is Major J. Petrie, and the new Vice-Chairman is Mr. H. J. Deane. A hearty vote of thanks has been accorded to the outgoing Chairman, Mr. H. D. Searles-Wood, and the outgoing Vice-Chairman, Sir Henry Tanner.

MEETING IN FEBRUARY.

The 109th ordinary general meeting will take place at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, February 23, 1922, when Mr. H. Kempton Dyson (Member) will read a paper entitled " What is the use of the Modular Ratio? "

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *FEBRUARY, 1922.* **No. 2.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

The Council desire to thank those many members who have written to congratulate them upon the inception of the *JOURNAL*, and they also desire to remind members that it is hoped that the pages of the *JOURNAL* will be used as a medium for inter-communication between members.

Provincial members are invited to contribute to the discussions upon papers read; and the Editor will welcome letters (which should not exceed 200 words in length) dealing with topical matters of interest.

REINFORCED CONCRETE PIERS AND MARINE WORKS.

BY W. NOBLE TWELVETREES, M.I.Mech.E.,
M.Soc.C.E. (France),

Ordinary concrete has been employed for very many years in the construction of piers, dock and harbour works, and reinforced concrete has been similarly used for nearly a quarter of a century in this country. During that period some 350 maritime and kindred structures are said to have been completed on the Mouchel-Hennebique system alone, and if similar statistics were available as to the works executed in accordance with other systems of reinforced concrete, the number mentioned would be very largely increased.

While mass concrete or large blocks of plain concrete may be advantageously employed in important works where the engineer finds solid rock or a firm stratum of earth capable of supporting heavy masses of concrete, it generally happens that alluvial soil is encountered in tidal rivers and estuaries. Therefore numerous wharves, jetties, piers and other marine structures have been built of iron, steel and timber, with the object of reducing as much as possible the imposition of useless weight upon strata of low bearing power. The inevitable corrosion of iron and steel, and the decay of timber render these materials particularly unsuitable for employment in construction situated partly below and partly above water level, to say nothing of the injury caused to timber by destructive sea worms.

Reinforced concrete, when properly designed and carefully applied by experienced contractors, is free from these drawbacks. At the present prices of materials, it is probably little, if at all, more costly than timber, and the ultimate economy resulting from its exceptional strength, durability and fire-resisting properties, gives reinforced concrete a clear advantage over the other materials mentioned.

Although maritime structures constitute only one of the many classes of construction to which reinforced concrete is applied, the subject is a very large one, and it is impossible in this paper to deal with it except in a very general way,

tracing the progress made during the past twenty-four years, and giving brief particulars of works representing different types of maritime structures.

The first reinforced concrete maritime work in this country is a retaining bank constructed in 1897 at Southampton for the London & South Western Railway Company, the bank being held up by a row of piles driven 17 ft. into hard sand and shingle, and finished at the top by a continuous coping.

In 1899 the first jetty in the kingdom was built at Woolston, in Southampton Harbour. After having been under heavy traffic for 22 years, this jetty is still in perfect condition, having cost practically nothing for maintenance.

Another early work, dating from the year 1899, is a wharf with reinforced concrete decking, at Prince's Dock, Liverpool; other works in the same port having been commenced during the following year.

The first maritime work embodying the cylinder pier system, invented by the late Mr. L. G. Mouchel, was constructed in 1900 at Dagenham, on the Thames, and in the same year the Admiralty commenced a long series of reinforced concrete works at Devonport and elsewhere.

The economy and structural advantages of reinforced concrete having been demonstrated by these and other pioneer works, the next period of five years witnessed the awakening of general interest in the new method of construction among dock and harbour authorities, and by the end of 1906 many maritime structures had been completed in the West and South of England, on the Thames and the Tyne, and in Scottish and Irish seaports.

From 1906 to the present date, the employment of reinforced concrete in maritime structures increased very greatly, so that examples of such construction are now to be found in all the chief seaports and tidal rivers of this country. Among such works are included: quays, wharves, jetties, piers, pier roundheads, and approach viaducts, shipbuilding berths, slipways, landing stages, dolphins, crane foundations and roads, breakwaters, sea walls, groynes, pontoons, caissons, barges, steamships and other floating structures.

The Author now proposes to give brief particulars and illustrations of a few typical examples of the foregoing classes of maritime structures.

QUAYS, WHARVES, JETTIES AND PIERS.

As originally designed, reinforced concrete quays, wharves, jetties and piers were founded on piles driven into

the bed of the sea, harbour, or river, the top of each pile being trimmed off a little above low water level. Pillars were then carried up to deck level, in continuation of the piles, being braced by horizontal and diagonal members, and connected with the longitudinal and transverse beams and continuous slab constituting the decking. Although satisfactory in places where the depth of water is comparatively small, this type of construction is apt to be somewhat lacking in rigidity, owing to the absence of subaqueous bracing between the piles, in situations where there is a considerable depth of water at low tide.

This objection was overcome by the cylinder pier system patented by the late Mr. L. G. Mouchel. Each pier is formed by driving a group of piles into the bed of a river or harbour, sinking a cylindrical shell over the projecting portions, and filling the interior with concrete, suitably reinforced. The shell, usually ranging from 3 ft. to 6 ft. in diameter, is precast in sections of convenient length, the bottom section being provided with a cutting edge enabling it to penetrate into the soil. The sections are reinforced with longitudinal and circumferential rods, and the edges are moulded so as to form socket and spigot joints. Such piers possess great strength, rigidity, and stability, and have been used with much advantage in the construction of many piers and wharves for the accommodation of large steamships.

A further advance is represented by the method introduced by Messrs. Mouchel & Partners wherein precast units are applied to the construction of subaqueous bracing for structures formed either with piles or with cylinder piers. In cases where this form of bracing is adopted in conjunction with cylinder piers, triangular frames are moulded on predetermined sections of the shells, the outer end of the frames being afterwards cemented into recesses formed in adjacent piers or elsewhere.

A fourth method of construction, particularly suited to exceptionally massive piers or jetties, is one embodying the use of large caissons, capable of being floated into position, sunk on prepared beds, and afterwards filled up with stone or rough concrete, as described in a later section of this paper.

Having regard to the very large number of works in existence, and the extremely varied uses to which they are applied, the Author has not found it an altogether easy task to select examples for illustration and description, but he has chosen a few representative structures typical of the different methods of design enumerated above.

Woolston Jetty.

The jetty incidentally mentioned as having been built in the year 1899, is still one of the best examples of jetty construction extant, although surpassed in size by numerous works since completed. Carrying a 35-ton crane and calculated for a distributed superload of 5 cwt. per square foot, the jetty projects 136 ft. into the harbour, and has a frontage of 100 ft. The jetty proper has a width of 46 ft., and is connected with the shore by an approach viaduct.

This pioneer structure remains as a permanent justification of the claims made for reinforced concrete at a time when considerable doubt was expressed by British engineers as to the reliability of that material. It has cost practically nothing for maintenance beyond some repairs in 1902, when a steamship collided with the jetty.

Yorkhill Dock Wharf, Glasgow.

The wharf forming the east end of Yorkhill Dock, Glasgow, was the first reinforced concrete structure of its class to be built on the Clyde. It is founded entirely on piles, those in the back row being connected by sheet piling terminating a little above water level. The superstructure consists of pillars built up from the foundation piles, securely braced, and finished by decking which comprises longitudinal and transverse beams and a continuous slab, 6 in. thick. It may here be mentioned that in a paper read to the Institute in 1917, Mr. William Cleaver, of Port Talbot Docks, referred to the fact that the ignition of oil floating on the surface of the water in docks was frequently the cause of serious fires, and constituted a strong argument in favour of reinforced concrete for the construction of wharves, quays and jetties.* The wisdom of his warning was emphasised a few months later by an outbreak of fire in Yorkhill Dock, where oil on the surface of the water was set alight, probably by the dropping of a redhot rivet, and started a fire which proved very difficult to fight, and caused serious damage. Most of the quays were of timber, which offered ready fuel for the flames, the reinforced concrete portion sustaining no damage beyond the charring of the timber fenders along the front.

War Office Pier, Shoeburyness.

Built in 1909 for the shipment of heavy gun mountings, the War Office Pier at Shoeburyness is a good example of braced construction. It is 360 ft. long by 26 ft. wide, for a

* "Transactions & Notes," Vol. viii, 5th paper, p. 29.

length of 250 ft. from the shore, thereafter widening out so as to give a frontage of 100 ft. at the end, where a 20-ton crane is installed on special foundations supported on reinforced concrete piles. The pier carries railway lines and was designed for a test load of 48 tons.

Dunston Jetty, Newcastle-on-Tyne.

One of the most useful jetties in the country is that now practically forming the river front of the extensive granaries, flour mills and manufactories of the Co-Operative Wholesale Society at Dunston. When built some 20 years ago, the original portion of the jetty was nearly 200 ft. away from the old quay wall, and stood in the River Tyne with a wide channel between it and the shore, except at one end where land was reclaimed for building purposes. A few years later more land was reclaimed, the jetty being widened and extended to a total length of over 800 ft. Extensive quays have also been built in front of the old river wall. As the soil is of extremely unstable character, reinforced concrete piles have been employed with much advantage in the foundations of the various buildings, as well as for those of the jetty and quays.

Quay and Jetties at the Armstrong Naval Shipyard, Walker-on-Tyne.

Reinforced concrete has been employed for over 15 years at the Elswick Works of Messrs. Armstrong, Whitworth & Co. in the construction of numerous works, including shipbuilding berths, wharves and jetties of considerable importance. When the establishment of the Walker shipyard was necessitated by the greatly increased size of modern warships, reinforced concrete was adopted as the material of construction for a long deepwater quay, and three jetties required for four of the largest shipbuilding berths. The quay is 2,133 ft. long, and is used as a fitting-out wharf. This is one of the longest reinforced concrete quays hitherto built, and is an excellent example of the framed construction so largely adopted in Hennebique practice. The decking carries tracks for a 30-ton electric crane, and the end next to the shipbuilding berths is finished with a substantial roundhead. The three jetties vary in length from 200 ft. to 350 ft., all of them being 30 ft. wide. The construction is marked by great simplicity, and although the various members are relatively slender, the monolithic character of the work, and the efficient nature of the bracing, furnish an ample guarantee of strength, stiffness and stability. The jetties were calculated for a dead load of 6 cwt. per square

foot in addition to the moving load of two 10-ton travelling cranes and a 5-ton electric tower crane.

Coaling Wharves and Jetties, King's Dock, Swansea.

A somewhat unusual feature in the general design of the coaling arm of the King's Dock, Swansea, is the adoption of projecting jetties, whereby the quay accommodation has been very largely increased. On the north side a frontage of 1,000 is occupied by the Great Western Railway coaling wharf and coal tip jetties, the latter being served by reinforced concrete viaducts; and on the south side a frontage of 1,790 ft. has been occupied by the Midland Railway wharf, tip jetty and viaduct, and by two coal tip jetties and viaducts constructed for the Swansea Harbour Trust.

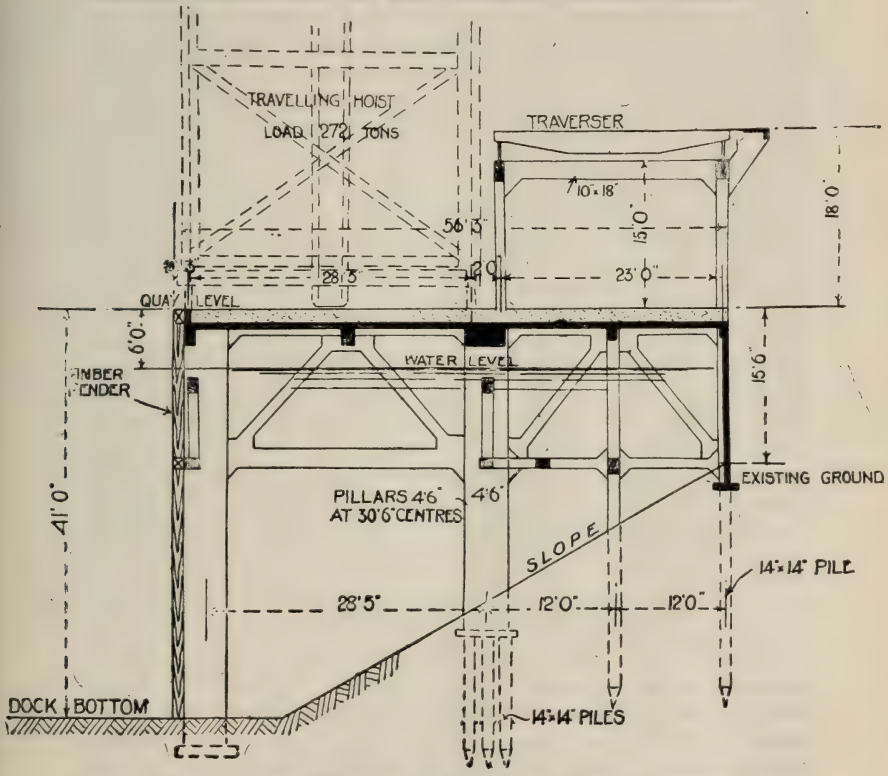


Fig. 1.—Great Western Railway Coal Wharf, Swansea.

The wharves and jetties are of exceptional strength and solidity, and rise to a height of 41 ft. above the dock

bottom. It was originally intended to build them in ordinary masonry, and the scheme then prepared embodied a series of longitudinal arches, extending some 60 ft. back from the quay front. Owing to the uncertain nature of the subsoil the cost of executing the works in this manner was found to be quite prohibitive, and the engineers came to the conclusion that reinforced concrete would be more economical, and at the same time more reliable than any other form of construction.

The railway viaducts serving the wharves and jetties are not maritime structures, strictly speaking, and moreover do not differ materially from many similar viaducts constructed in other seaports. The wharves and jetties, however, are of somewhat unusual design, and were constructed in a manner which is seldom possible in the case of dock and harbour work. Owing to the heavy loads involved by the employment of fixed and travelling coal hoists, the front portion of each structure was built up with massive pillars, 4 ft. 6 in. square and spaced 30 ft. 6 in. apart longitudinally, and as the work was executed before the admission of water into the new dock basin, these pillars and all parts of the work afterwards destined to be permanently submerged were constructed with the same facility as in the case of reinforced concrete land structures. The pillars in the front row are provided with extended bases of the usual type; those in the second row having extended bases which are supported on groups of 14 in. square piles. The rear portion of each wharf and jetty is founded on piles continued up in the form of pillars 14 in. square in cross section. All these members are securely braced and completed by decking in the usual manner. The Great Western Railway wharf includes a traverser gantry 475 ft. long serving two travelling coal hoists running along the front, loaded trucks being transferred from the approach viaducts to the traverser and delivered thence to the hoists.

Dock Wall Jetty and Dolphins, Cork.

At Passage West, Cork, reinforced concrete was applied recently to the strengthening of a quay wall 660 ft. long, and the construction of a jetty and three dolphins in connection therewith, the former 180 ft. long by 18 ft. 6 in. wide, and the latter each 45 ft. by 18 ft. 6 in. in plan, all these structures being reinforced with Indented bars. The quay wall was strengthened by driving piles along the front at intervals of 7 ft. 6 in. apart, and filling the intervening

spaces with plain concrete, the piles being tied back to anchor beams extending the whole length of the quay. The jetty and the dolphins are constructed of piles, pillars and bracing members, finished with decking of the usual type, the whole in monolithic connection.

Jetty at Southampton.

Another work embodying Indented bars is a jetty at Southampton, 100 ft. long with a maximum width of 30 ft., the structure consisting of piles, pillars, bracing and decking. Railway lines are carried down the jetty, at the end of which a crane is supported.

Lady Lloyd Pier, Karachi.

A third example of Indented bar work is the Lady Lloyd Pier, Karachi. The pier is 1,300 ft. long with a minimum width of 15 ft. 9 in. clear, and has a T-head approximately 60 ft. long on which a pavilion with reinforced concrete roofing will be erected. Owing to the slope of the foreshore the decking is carried at six levels, four short sections at the beginning and two longer ones at the end, each being joined to its successor by a flight of steps. The substructure consists of piles and piers upon which framed construction of the usual character has been added.

New Fish Dock, Fleetwood.

An interesting operation carried out by the Lancashire and Yorkshire Railway Company during recent years was the conversion of an existing timber pond at their Fleetwood Docks into a new dock for the fishing industry. An important part of this project was the construction of reinforced concrete quays along all four sides of the basin. The new quays are founded on piles driven into the ground at the bottom and along the margin of the dock, most of the quays consisting of trestles each including three piles and pillars, connected longitudinally by horizontal bracing and by the deck system of beams and slabs, as shown in Fig. 2. The spaces between the front foundation piles were filled by sheet piling to form a watertight barrier. In addition to the resistance afforded by the monolithic nature of the construction, the stability of the quays is further assured by reinforced concrete land ties securely anchored to blocks of plain concrete embedded in the ground opposite every alternate trestle. Three sides of the new dock are provided with fish

quays having an aggregate length of 1,860 ft., the fourth side being occupied by a coal quay, 680 ft. in length.

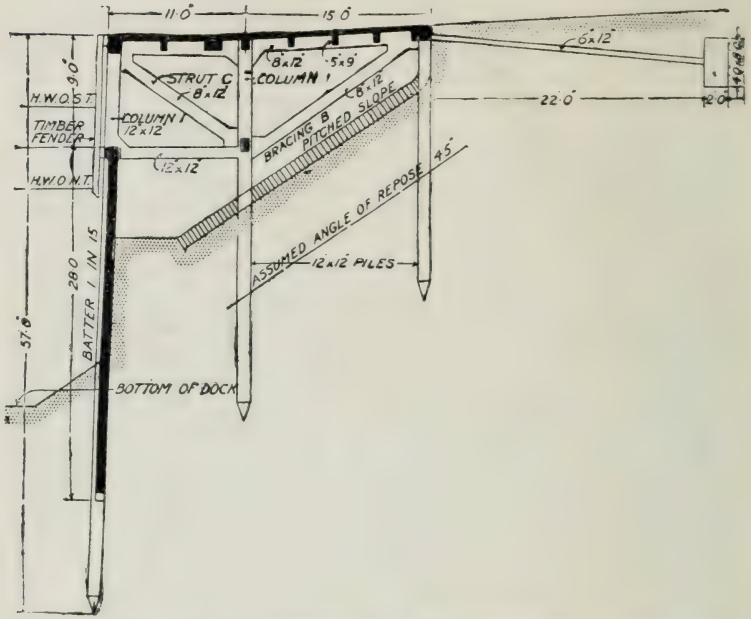


Fig. 2.—New Fish Dock, Fleetwood.

Typical Section, Illustrating the Construction of the Fish and Coal Quays.

Oil Fuel Jetties.

One of the first oil pipe jetties designed for the supply of fuel to steamships is that which was constructed in 1911 at Gosport for the Admiralty. This jetty, built out across a bank of soft mud into deep water, is 981 ft. in length, by 25 ft. in width up to the head, which is 63 ft. 6 in. wide. Another oil jetty, constructed at Avonmouth Docks in 1918, embodies some interesting examples of precast bracings combined with ordinary reinforced concrete piles and other members. The latest example is the oil fuel island jetty built for the Anglo-American Oil Company at Brixham. The structural features of this jetty are at once interesting and unusual. Briefly described, the work consists of three strongly framed dolphins, rectangular in plan, connected by

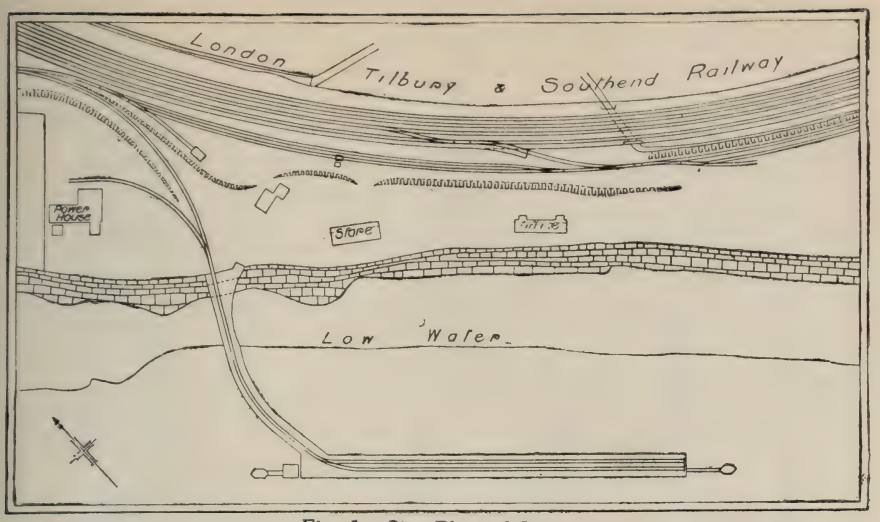


Fig. 5.—Site Plan of Jetty.

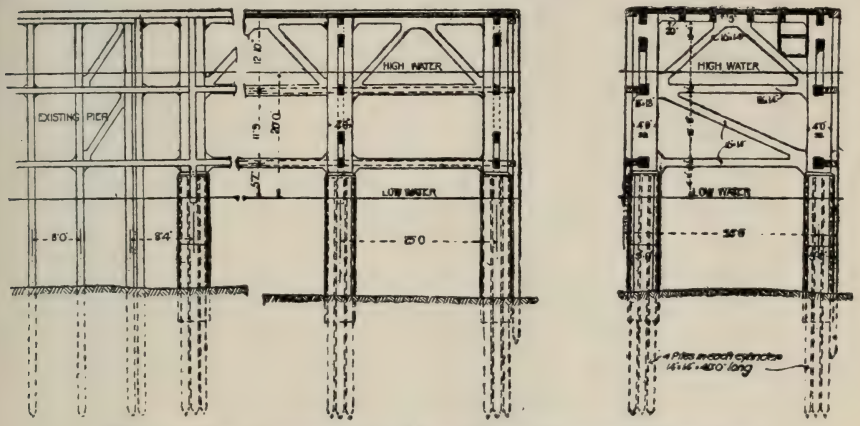


Fig. 6.—Part Elevations of Old and New Portions of Jetty, and Cross Section of New Portion.

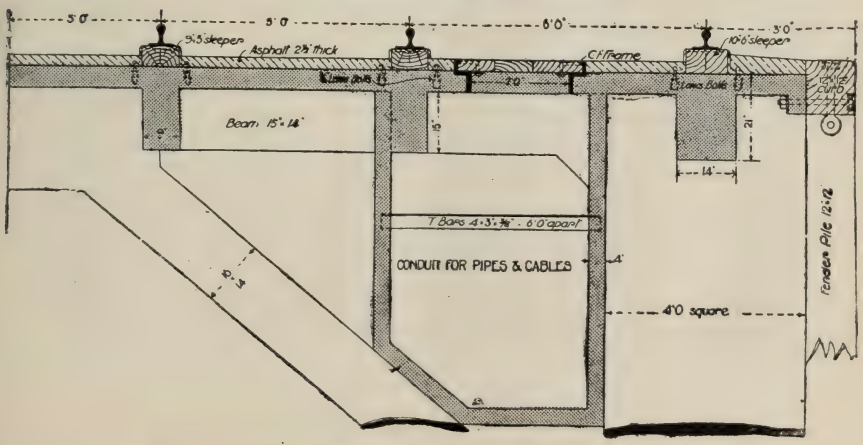


Fig 7.—Section showing Conduits for Pipes and Cables.

construction of the kind generally to be found in reinforced concrete wharves, and the whole connected by continuous decking, at a height of 10 ft. 6 in. above H.W.O.S.T., and 24 ft. above L.W.O.S.T. The front of the jetty is at a distance of 166 ft. from the harbour wall, and is 258 ft. in length. As the deck level is 51 ft. 6 in. above the bottom of the harbour, and in view of the heavy pull exerted on the bollards by ocean steamships of the largest size, the three main dolphins are of exceptionally powerful construction. They are founded on square piles driven in groups of five, each group being enclosed in a rectangular casing similar, except in shape, to the cylinders employed in Mouchel pier construction. The casings are 6 ft. 6 in. square, and in addition to being strongly braced in every direction, the solid piers are connected by precast slabs fitting into grooves and forming watertight partitions for the retention of rock filling. The square piers terminate 3 ft. above low water level, and are finished by capping on which the superstructure was erected. The portions of the jetty between the dolphins are founded on 14 in. square piles, and in them, as well as in the superstructure over the dolphins, precast units have been freely employed. The oil pipe is laid on a cantilever platform along the back of the jetty, and is carried along a light approach viaduct to the shore, where it is connected with the oil reservoirs and pumps, the total length of the pipe line being more than 3,000 ft. The pipe is of 10-in. internal diameter, and is capable of delivering nearly 260 tons of oil an hour.

Quay Widening Works.

Another most useful application of reinforced concrete is in the execution of quay widenings. Many works of this class have been executed in the principal seaports of the United Kingdom, some embodying ordinary pile construction and others representing examples of Mouchel cylinder pier construction. A simple but extremely useful example is to be found in Victoria Dock, Leith, where the south quay was originally formed from an old breakwater, the face of which sloped so that the deck of any vessel moored alongside was about 8 ft. from the edge of the quay. With the object of doing away with this serious inconvenience the quay was widened and straightened by driving reinforced concrete piles in front of the old wall, bracing them by longitudinal walings and tying them back into the existing

masonry, the ties near quay level being connected with concrete anchor blocks spaced 3 ft. apart. (See Fig. 3.)

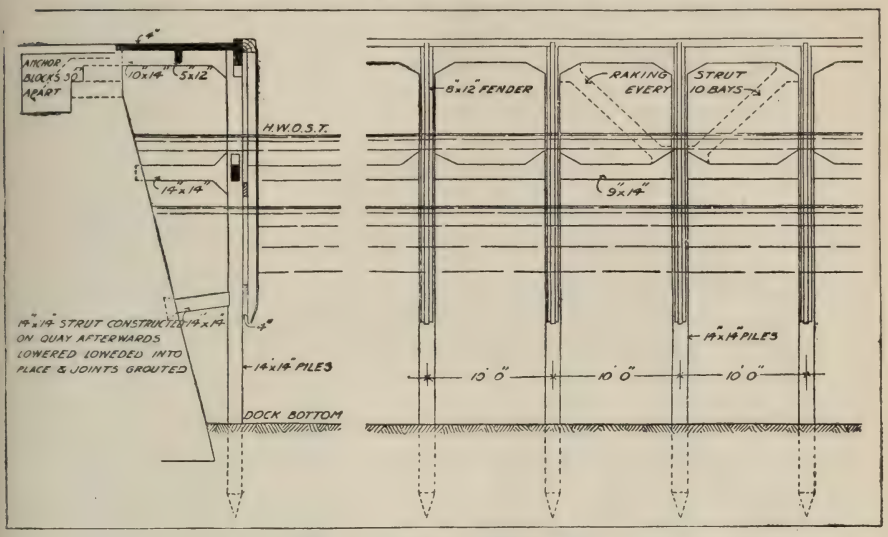


Fig. 3.—Victoria Dock, Leith.
Details of Quay Widening Work.

Several important examples of quay widening works are to be found in the docks of the Port of London Authority, the widenings varying from 15 ft. to over 55 ft. The north quay at the Western London Dock was widened 15 ft. by

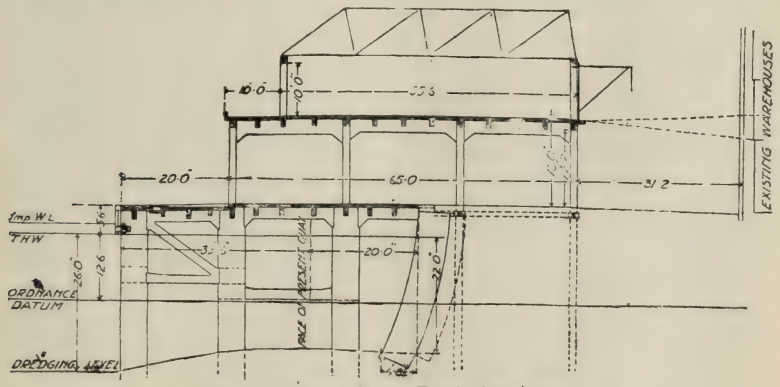


Fig. 4.—West India Docks, London.
North Quay Widening and New Shed in Import Dock.

pile construction, braced and tied back to the old wall by means of precast frames. Several other widenings in the docks of the same authority embody Mouchel cylinder piers with precast bracing frames, moulded at the same time with predetermined sections of the pier casings. Fig. 4 is a cross-section of the widened north quay and new shed in the Import Dock of the West India Docks.

Dagenham Jetty, Essex.

The coaling jetty built at Dagenham twenty years ago was the first maritime work executed on the Mouchel cylinder pier system. It is 780 ft. in length by 35 ft. wide, and the T-head carries four transporter cranes weighing 60 tons each. The height from the river bed to deck level is 78 ft., the rigidity of the construction being amply assured by the cylinder piers.

Purfleet Jetty and Viaduct.

The original portion of the structure at Purfleet was completed eighteen years ago, and the remainder some eight years later. The jetty now has a river front 586 ft. in length, the width being 34 ft. over all. The approach viaduct is 300 ft. long, and includes a bowstring girder bridge of 60 ft. span. The original portion of the jetty, and the viaduct down to low water mark, is founded on piles, 14 in. square, the remainder of the viaduct and the newer portion of the jetty representing cylinder pier and precast frame construction. Various structural details of the jetty and viaduct will be found in Figs. 5 to 8. The remarkable strength, cohesion and rigidity of the jetty have been demonstrated by three serious collisions, two in the year 1907 and one in 1917.

Collision No. 1 was due to derangement of the steering gear of an 8,000-ton steamer, which followed a straight course of fully a mile and crashed into the jetty at high speed. As stated by Mr. C. S. Meik, at the Engineering Conference of 1907, the only damage was the destruction of seven or eight piles and about 20 square feet of the decking. Mr. Meik also stated that after the accident a thorough examination was made, the entire absence of cracks showing the perfect resilience of the piles. Mr. J. H. Anderson, of the Coal Owners' Association, who witnessed the collision from the approach viaduct, has stated that when the steamer crashed into the pier it penetrated a few feet and apparently bent over part of the structure, which quickly righted itself

PURFLEET JETTY, ESSEX.

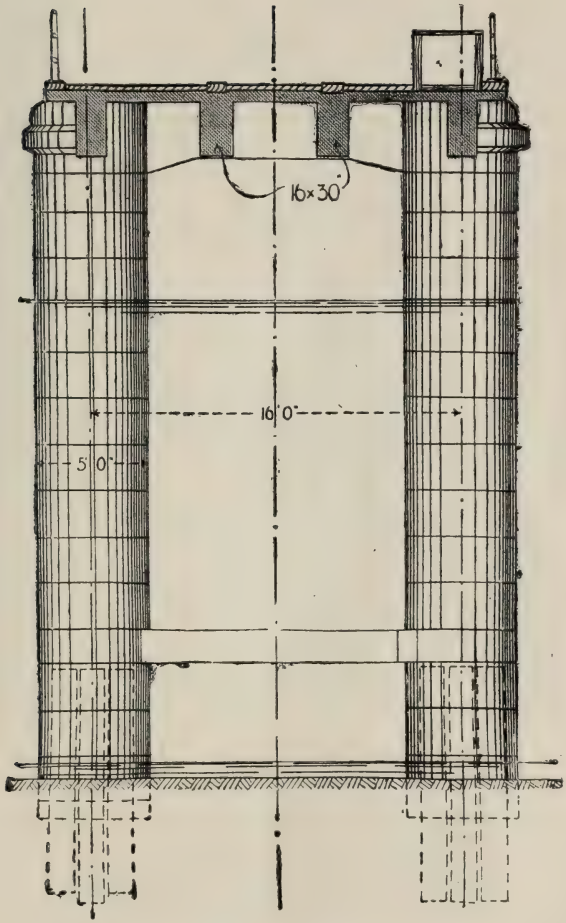


Fig. 8.—Section of Approach Viaduct.

and shot the vessel back into the river as if it had been expelled from a catapult.

Collision No. 2 was also due to defective steering gear in a large steamship, and took place at the approach viaduct. The damage in this case was slight, a fact which is attributed by Mr. Anderson to the circumstance that the steamer first struck one of the substantial 5 ft. diameter cylinder piers.

Collision No. 3, in October, 1917, is described by Mr. Anderson as being without doubt the heaviest test given to the jetty. On this occasion a steamship laden with 9,000 tons of cargo was diverted out of its course by some navigation difficulties, and although the captain tried to avoid the jetty by increasing the speed of his vessel, the collision took place with a terrific crash, the steamer being shot back into the river, where it was picked up by tugs and got safely into dock. On examination the next day, the jetty was found to be in perfect alignment, the only perceptible damage being the penetration of the decking and other members for a depth of about 13 ft., without destroying the cohesion of the work around the gap, or impairing the structure as a whole. This point is forcibly illustrated by the fact that on the bay next to the damaged part stood a hydraulic crane weighing 160 tons, and rising to a height of 100 ft. above the deck. It speaks well for the rigidity of the work that this crane was not toppled over or injured in any way. In this collision and in the two previous collisions the steamships are stated to have suffered far more seriously than the reinforced concrete work.

Alexandra Wharf: Newport, Mon.

Constructed in 1905 for the Alexandra Docks and Railway Company, the Alexandra Wharf at Newport, Mon., is 400 ft. long and projects 47 ft. into the water. The front is founded on cylinder piers, the rear portion being of piles and pillars braced in the usual manner. The wharf is equipped with three railway tracks, each designed for a rolling load of 80 tons, and a fourth track for the operation of several 100-ton hydraulic cranes. About five years ago an accident occurred which is of interest as showing the behaviour of reinforced concrete under sudden and unexpected shocks. The mishap was caused by a train of goods trucks which was accidentally engaged with a hook at the end of a loading chain hanging from one of the cranes and when the train was drawn away the crane was pulled over, falling sideways upon the decking, while the heavy

counterweight fell near the crane. The only damage done to the reinforced concrete construction was the cracking of one secondary beam and one panel of the deck slab. Examination demonstrated the interesting facts that the reinforcement, embedded more than ten years previously, was perfectly free from corrosion, and that the bond between the concrete and the steel was not impaired by the severe strain, the bars being stretched in the middle, and the ends showing no sign of displacement.

Wharves and Jetties at Port Talbot.

Several examples of maritime structures at Port Talbot have already been discussed in a paper read to the Institute by Mr. William Cleaver, the engineer to the Port Talbot Docks & Railway Company, who has fully described the coal tip jetties and other reinforced concrete works executed under his direction, and called attention to their advantages in respect of resistance to fire and to sudden shocks. Four mishaps at Port Talbot detailed by Mr. Cleaver may appropriately be very briefly summarised in the present paper, as follows:—(a) A railway truck falling from a height of 20 ft. upon the lower deck of a tip jetty caused no damage at all; (b) Three sailing ships, having broken loose, were brought up by colliding with a tip jetty, the plates of the leading vessel being badly dented without affecting the reinforced concrete structure, although the steelwork of the hoist was injured to some extent; (c) A 4,000-ton steamer crashed at fairly high speed into the weakest portion of a wharf, bursting through a beam and penetrating about 6 in. into the decking beyond, without causing even a crack outside a radius of 3ft. from the point where the ship's stem entered; (d) A 4,000-ton steamer collided with a wharf, and the propeller got jammed between a reinforced concrete pile and some timber fencing; the timber work was badly damaged, the propeller shaft was broken off, and the reinforced concrete was slightly damaged. The costs of repairs were £800 for the steamship, £42 for the timber, and £8 for the reinforced concrete, these figures speaking for themselves.

Tilbury Deepwater Jetty.

As this work was visited by members of the Institute last year it is unnecessary to occupy space in this paper by giving details of the jetty at Tilbury. Commenced at the beginning of the war, the jetty has recently been completed. Measuring 1,000ft. in length by 50ft. in width, and connected with the shore by an approach viaduct 900ft. long,

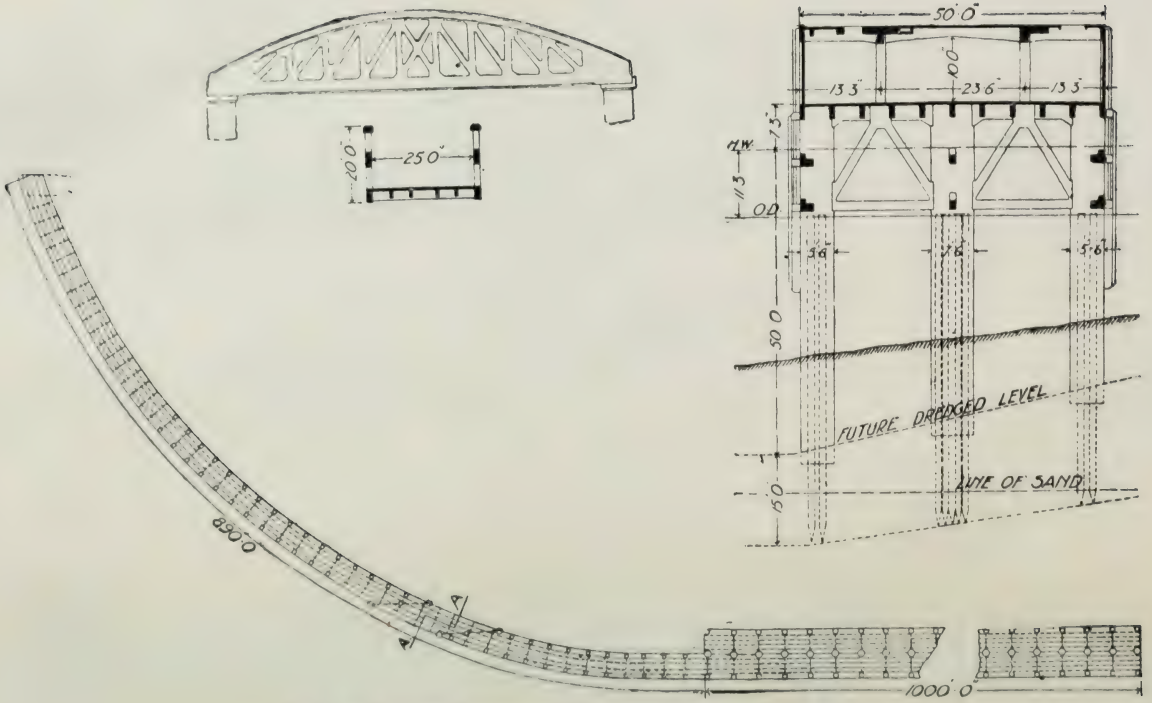


Fig. 9.—Tilbury Deepwater Jetty and Approach Viaduct.

including a bowstring girder bridge of 100ft. span, the jetty has two decks, the space between them forming a transit shed. (See Fig. 9.) The structure is carried on Mouchel type cylinder piers with precast bracing frame, the piers being sunk 2ft. into the bed of the river and securely anchored by clusters of piles embedded in the core of the cylinders. Built parallel with the shore outside Tilbury Docks, the new jetty enables large steamers to discharge and ship cargo simultaneously and proceed on their voyage with a minimum loss of time, thus complying with the condition brought about by the development of Tilbury as a port of call by many lines of British and foreign steamships.

CAISSONS AND PONTOONS.

One of the most remarkable uses of reinforced concrete is that represented by the construction of floating structures of various kinds, including caissons, pontoons, floating docks, barges, sailing vessels, motor boats and sea-going steamships.

The subject of concrete shipbuilding has already been discussed at previous meetings of the Institute, and for this reason, as well as in view of the fact that the industry is in a state of suspended animation at the present time in this country, the Author here confines attention to structures which, although capable of floating in water, are only intended to float while they are being transported to predetermined positions, where they are sunk and secured in place to form permanent construction, either partly or entirely below water level.

Probably the most interesting structure of the kind ever built in reinforced concrete is that known as the "Batterie des Maures," a large caisson which has now found a permanent resting place in the Bay of Hyères, on the Mediterranean, where it is employed by the firm of Schneider & Co. for testing torpedoes supplied to the French Government. This huge structure was built on shore, launched, towed out to sea and sunk on a levelled bed in deep water. It consists essentially of a caisson, above which is a superstructure of two storeys surmounted by a spacious deck. The caisson is over 50ft. high by 77ft. long by 45ft. wide at the base, and 65ft. long by 35ft. wide at the top. The outer walls are 6in. thick, and the interior partition walls 4in. thick. The superstructure rises about 20ft. above the top of the caisson, and at one end an observation chamber projects 17ft. as a cantilever. The sections reproduced as Fig. 10

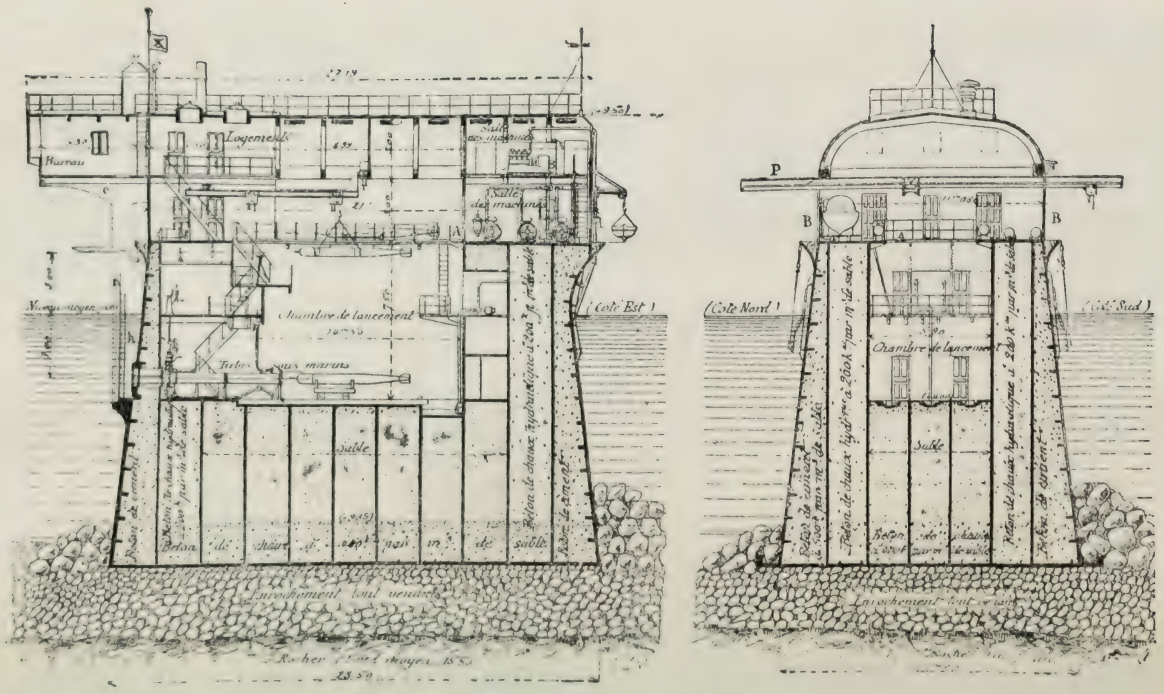


Fig. 10.—"Batterie Les Maures," Hyères.
Longtitudinal and Cross Sections.

illustrate the main features of the construction as well as the general arrangement and equipment of the interior.

The battery comprises a large chamber for the reception and discharge of torpedoes, an observation chamber for watching the course of the torpedoes, well equipped workshops, and a generating room where electricity is generated by dynamos driven by internal combustion motors, for the supply of current to air compressors, cranes, pumps and other appliances. Living rooms are also provided for the resident staff, as well as a landing stage, ladders, signalling apparatus and other auxiliaries.

The caisson was built in a dry dock at La Seyne, near Toulon, the work of constructing and placing it upon its bed being divided into three parts: (1) construction in the dock, (2) launching and towing to the site prepared, (3) sinking by means of ballast and protecting the base by a surrounding bank of large blocks of stone.

After the work of concreting had been completed, the top of the caisson was temporarily decked for the convenience of those who had charge of sinking operations. A warp of 10 cm. diameter passing twice around the caisson, provided with padding at the corners and held in place by cordage, was attached to a cable of 12 cm. diameter, the other end of which was made fast to two steam tugs arranged tandem fashion. Having a displacement of 2,600 metric tons and drawing 8 metres of water, the caisson was towed through the sea for a distance of about 40 kilometres, the passage occupying fifteen hours. The position of the foundation bed prepared in advance was marked by buoys, and when the caisson had been correctly aligned in place, it was sunk by admitting water to the interior compartments provided for the purpose, these compartments being afterwards filled with concrete. The blocks of stone forming the outer protection were deposited around the caisson, and the work of completing the superstructure was then taken in hand.

The stability of a work such as the Batterie des Maures constitutes a difficult problem, for it is practically impossible to determine precisely the horizontal resultant of the forces due to the action of the waves in rough weather. Therefore, the monolithic nature of the reinforced concrete construction is a great advantage, as the walls and other parts are thoroughly consolidated and rendered capable of withstanding forces which might cause injury to a structure built in accordance with ordinary methods. So far as stability is concerned, it may be pointed out that an ample assurance of

safety is provided by the fact that while the total weight of the battery is 9,000 metric tons, the weight of water displaced is only 3,700 metric tons, leaving a net weight of 5,300 metric tons. Further, it is stated that the structure is capable of offering 50 per cent. more resistance to overturning moment than the maximum resistance for which provision need be made anywhere on the coast of the Mediterranean.

Two other examples of caisson construction in pier and kindred works will now be cited, and although far less striking in appearance and object than that already described, they are nevertheless of considerable interest.

The first of these examples is furnished by the caissons employed a few years ago in the construction of a jetty 100 metres in length at Alexandria, in Egypt. Five caissons were used in the construction of the jetty, all of them 20 metres long by 8 metres wide from face to face, but having cantilevers on each side increasing the effective width to 9 metres. The first two caissons at the shore end were 5.40 metres high, and the last three, where there was a greater depth of water, were 7 metres high. All were divided by a central longitudinal partition, and by four intermediate transverse partitions.

The larger caissons have each a volume of 1,000 cubic metres, and weighed 400 metric tons when afloat the draught being about 2 metres. After being filled when sunk in place, each caisson became a monolithic mass with the weight of 2,500 metric tons, approximately. Such would have been the quantity of concrete it would have been necessary to hoist, transport and deposit in place to obtain the requisite stability if ordinary methods of construction had been adopted, and it will be clear that costly staging would have been needed for the execution of the work in that way.

By the adoption of reinforced concrete caissons, the total weight of material to be towed was comparatively small, and after the caissons had been sunk in line the top surface constituted a causeway along which the plain concrete filling was transported very readily and economically. The top of the caissons was constructed with curbs so as to retain a layer of sand for the reception of the granite sets forming the paving of the jetty.

The second example of caisson, or pontoon, construction is one furnished by the slipways laid down originally for concrete shipbuilding at Barrow-in-Furness. The water front of the shipbuilding yard in question is bounded by a quay wall instead of sloping down into the channel. Conse-

quently, it became necessary to make provision for supporting the outer end of the slipways. The requisite support might have been provided by piling or trestles, but it was thought preferable to employ reinforced concrete floating caissons or pontoons, which could be readily moved from one building berth to another as occasion required.

The pontoons were sunk in place opposite any berth by the admission of water into the interior. In order to float them for removal to any other berth, it was only necessary to drain off the water at low tide, and to tow the pontoons to their new position on the next rising tide.

SLIPWAYS AND SHIPBUILDING BERTHS.

A considerable number of works coming in this class have been executed in the United Kingdom, among them being the slipway for the Woolston floating bridge in Southampton Harbour; slipways at the shipyards of Messrs. J. S. White & Co., Ld., Cowes, and Messrs. Cammell Laird & Co., Ld., Birkenhead; shipbuilding berths for the Forth Shipbuilding Co., Ld., at Alloa; and numerous slipways and berths in the yards which were so actively engaged in concrete barge and ship building during the war. Various ingenious methods of constructing building berths were adopted in the concrete shipyards, one which was found very satisfactory in practice embodying the construction of concrete foundations taking the form of a pathway following the outline of the vessel to be built, with a centre strip wide enough to carry the keel blocks, which, together with the bilge blocks, were moulded in advance with two pieces of pipe extending from side to side, so that the blocks could be readily handled by the aid of steel bars passed through the holes. The blocks were built up to the required height with bedding pieces of felt between successive blocks, oak wedges being placed on the uppermost blocks, and on the wedges were laid bearers for the timbering of the hull.

LANDING STAGES.

Since the construction in 1904 of the landing stage for mail tenders at Plymouth by the Great Western Railway Company, kindred structures have been executed in reinforced concrete at Southampton, Gosport, Chichester, Herne Bay, Newlyn and various other places. The most interesting of these landing stages is one at Clevedon for use by passenger steamers plying between that seaside resort and large centres of population on either side of the Bristol Chan-

nel. Situated at the end of an iron pier 850 ft. in length, the landing stage is a framed structure of reinforced concrete, 100 ft. long at the sea bed, and 95 ft. long at the top, by 30 ft. wide at the sea bed, and 12 ft. wide at the top, by 53 ft. in height from the sea bed to the topmost decking. It is founded upon twenty-two piles, driven into the marl to solid rock. (See Fig. 11.)

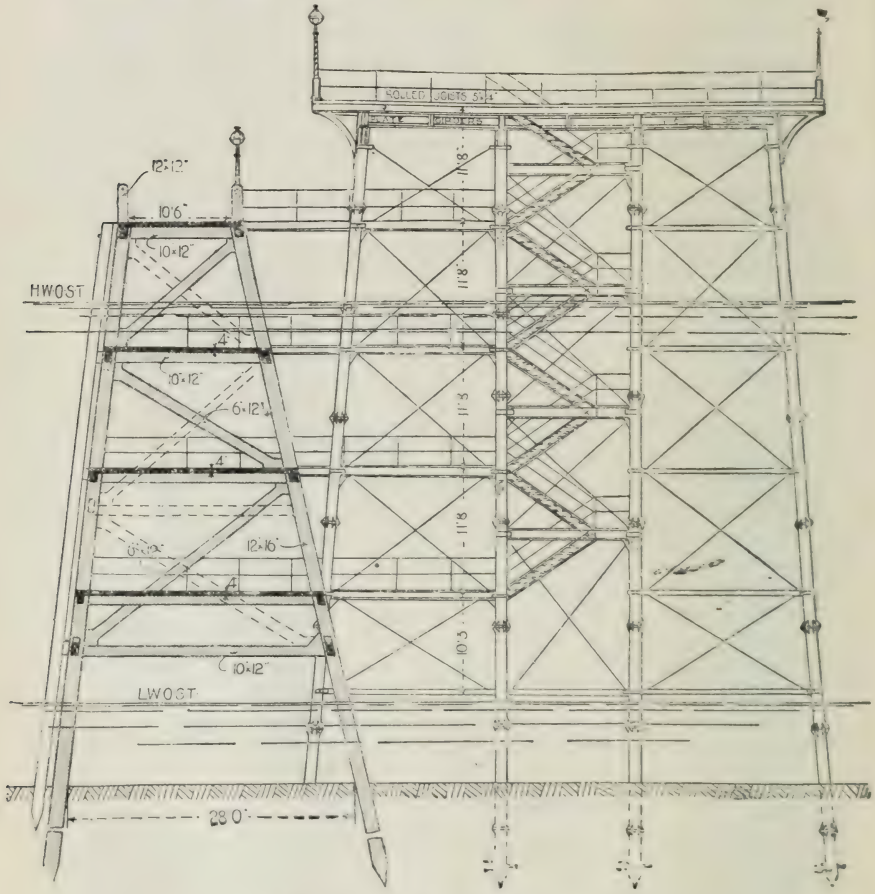


Fig. 11.—Landing Stage, Clevedon.
Sections of Pier Head and Reinforced Concrete Landing Stage.

After having been driven, the top of each pile was a little above low water level. Upon the piles was built the framed

superstructure, consisting of pillars in continuation of the piles and connected by horizontal and diagonal bracing, as well as by the beams and slabs of the upper deck and the three landing platforms provided for convenience in embarking and disembarking passengers at any state of the tide.

In view of the fact that the tidal range is 49 ft. at the head of the pier, it will be understood that the work of the contractors was not unattended by difficulties, which were materially increased by bad weather. The wind frequently prevented the tides from ebbing to normal level for several days at a time, thus causing delays as well as anxieties.

In order to expedite matters, a good many of the members were moulded in advance on the pierhead, so that they could be quickly built in position. In spite of all precautions some minor troubles occurred. For instance, just after the decking of the second platform had been concreted, some of the green concrete was washed away by heavy seas. However, considering the dangers and difficulties encountered, it is satisfactory to know that the work was completed without any serious mishap or accident to the men employed.

During the severe storm of March, 1914, heavy seas broke over the landing stage without causing any injury, a fact worthy of note for the reason that since the storm of December, 1910, when the old landing stage was wrecked, Clevedon had not been visited by a gale of anything like equal violence.

COAST PROTECTION WORKS.

The chief objection to the timber groynes which are so largely used in coast protection, is the perishable nature of the material. Within recent years, this objection has been entirely overcome by the adaptation of reinforced concrete to groyne design under the Owens and Case patent. The new type of groyne is of very simple character, consisting of grooved reinforced concrete pillars fixed in holes excavated in hard soil, or grooved reinforced concrete piles driven into soft soil, reinforced concrete slabs being dropped into place between the pillars or piles. The uprights, in either case, are made of sufficient height to allow additional slabs to be inserted as beach material accumulates, and where necessary raking struts are applied to insure lateral stability.

When the beach has been built up to the required height, the groynes are completed by slabs rounded at the top to form a smooth coping along the entire structure.

Many groynes of this type have been built on the South Coast and elsewhere. The first cost is said to compare

favourably with that of timber construction, while the ultimate cost is very much less, owing to the permanence of the material.

A similar method of employing reinforced concrete units has been applied to the construction of sea walls. An example is furnished by a sea wall in the Solent, near the entrance to Southampton Water, the new work taking the place of an old clay bank protected by timber sheet piles. Owing to the heavy annual cost of repairs, it was decided to replace the bank by a permanent wall of reinforced concrete. The latter consists of grooved pillars, 9 ft. long by 14 in. square, spaced 7 ft. apart, built up from concrete foundations, between them being reinforced concrete slabs 12 in. wide, and ranging from 14 in. to 6 in. in thickness. The slabs are placed in the grooves and grouted so as to form a perfectly water-tight wall. A reinforced concrete counterfort is attached to the head of each pillar, the lower end being terminated by a foundation block of plain concrete. The top of the wall is finished by a coping about 6 ft. above high water level.

Reinforced concrete retaining walls and sheet piling have been employed very extensively in protecting the banks of tidal rivers and other navigable channels, but it is scarcely necessary to occupy time by giving details of such works. It may be of interest, however, to refer to the case of a sheet pile which was exhibited a few years ago by Mr. W. J. Taylor, the county surveyor of Hampshire, after having been exposed to the action of tidal waters for a period of some fourteen years. Having been drawn and carefully examined, both the concrete and the steel were found to be in good condition, the steel being perfectly free from any trace of corrosion.

While dealing with coast protection works, reference may appropriately be made to the reinforced concrete breakwater built by the Irish Board of Public Works to protect the fishing harbour of Passage East, near Waterford. So far as the Author is aware, this is the only example of a breakwater in the United Kingdom. The structure is 120 ft. long by 20 ft. wide, the sides being parallel for a length of about 80 ft., then converging to a point at each end. If this breakwater had to be constructed in the present day, it would probably be formed by sinking a caisson similar to those described in connection with the jetty in Alexandria Harbour. At the time when the breakwater was built, reinforced concrete engineers had not the advantage of possessing the extended experience which has since been gained as to the

possibilities of caissons and other structures capable of being floated, and the breakwater was constructed as briefly described below. The sub-structure consists of a cofferdam formed by driving sheet piles around the site, the whole being grouted to form a water-tight wall. The piles varied in length between 15 ft. and 30 ft., according to depth of the harbour bed, and the top of the cofferdam was 3 ft. above low water level. The interior was filled with stone and rough concrete to give the requisite stability to the cofferdam, on the top of which was built a superstructure consisting of reinforced concrete pillars, walls, horizontal and diagonal bracing, longitudinal and transverse beams, and a continuous deck slab, the whole in monolithic connection.

An important example of coast protection work is represented by the promenade extension at Burnham, in Somersetshire, the total length of the extension being 1,500 ft., and its width 20 ft. from the roadway to the inside of the parapet. The original designs provided for cast-iron columns, steel beams and reinforced concrete decking, but it was afterwards decided to execute the whole in the last-mentioned material.

The promenade decking consists of a continuous slab 4 in. thick, carried on beams spaced 5 ft. apart, the latter supported at the landward end by a concrete wall, and at the seaward end by a continuous beam carried by pillars, 10 in. square, spaced 15 ft. apart, and terminated by extended bases which are founded on solid concrete blocks. In each panel of the deck slab an opening 3 ft. 6 in. long by

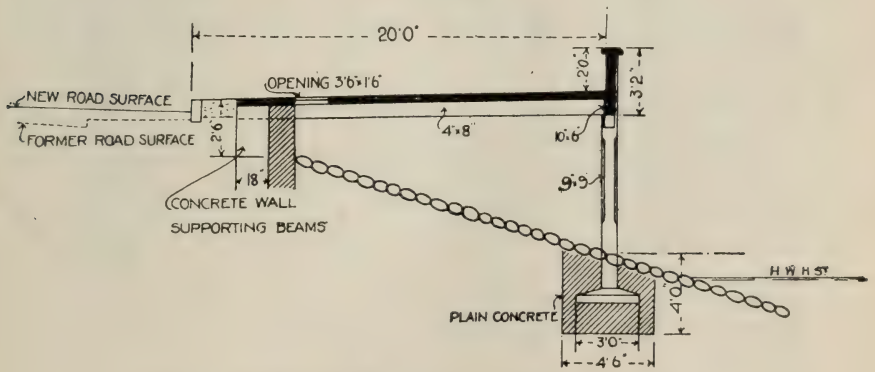


Fig. 12.—Marine Promenade, Burnham, Somerset.

1 ft. 6 in. wide is formed, protected by a grating, these openings being intended to permit the free escape of water and

compressed air in rough weather. The sea front of the promenade is finished by a parapet 2 ft. high by 3½ in. thick, with a coping 12 in. wide, this parapet being capable of acting with the continuous beam below the two members forming a girder 3 ft. 2 in. in depth. A typical cross-section is given as Fig. 12.

Constructed in 1911, the promenade was most severely tested by the high tides and heavy gales of March, 1914, when the strength and stability of the work undoubtedly saved the town from a serious inundation, such as that which occurred in 1910 before the execution of the protection work. In addition to enduring the tremendous force of the waves, the promenade was subjected to severe battering from fencing timbers and wooden seats that had been washed into the sea, and consequently suffered strains of far greater magnitude, and much more trying character than those contemplated by the designers. That the work was able to withstand these without material injury is a fact on which all concerned may be congratulated.

DISCUSSION ON PAPER.

Mr. C. H. Colson said the Institute ought to be very grateful indeed to Mr. Twelvetrees for the trouble he had taken in putting so many interesting facts before them. He (Mr. Colson) had had some considerable experience with reinforced concrete jetties and thought we were too ready nowadays to forget the conditions under which reinforced concrete had originally been taken up. When jetties of this material were first built we had nothing but steel or timber jetties. It was not, therefore, a question of whether reinforced concrete was going to be an absolutely permanent material, but of the extent to which its life would vary from that of the other materials. Since that time reinforced concrete had given its supporters many bad half hours, but it had justified itself. The main difficulty to be faced was that you had a solid material with, embedded in it, another material which might expand; you had also to take every precaution to prevent that material from corroding. In many cases, he was afraid, there was not sufficient cover left to the steel to really prevent its corroding. He did not in this remark refer to the first jetties, because he remembered very well that the first jetty that was built by his old friend Mr. Mouchel at Woolston was built with very great care. That had been more or less a show jetty, great care had

been taken in preparing the material; and ten or twelve years after its construction he had seen the lengths of the piles lying down below the jetty, and, upon having some of these broken up, he had found the steel quite as good as on the day when it was put into the concrete. But that fact did not prove to his mind, that it was correct to copy timber construction in reinforced concrete construction. In modern structures of reinforced concrete, bearing in mind the fact that we had to fight against corrosion, we wanted so to shape the members that the steel should be embedded as deeply as could possibly be managed. That, he submitted, could be best done by massing the members together, using large beams, and, he thought using cylindrical piles rather than copying a timber structure. Where the timber structure was copied, signs of cracks appeared at the angles of the piles where the reinforcement came nearest to the surface, and that, to his mind, tended to show that the circular or even octagonal form of pile was a better form than the square one; because in the square pile the angle was much more susceptible of damage than it was in the octagonal pile. But he wanted to go beyond that, and to suggest that we should not use these copies of timber structures, but should use large cylinders—with piles embedded in them no doubt—where we could keep the steel away from the action of the water. He was not sure that he quite agreed with Mr. Mouchel's reinforced concrete cylinders, because in them the conditions that he had laid down were not quite followed out. There you had a steel structure quite close to the surface of the cylinders, and he would like to ask Mr. Noble Twelvetrees what the result had been of experience on those cylinders? Had corrosion shown? Or had damage occurred where ships happened to touch? Coming to another subject, perhaps his experience of reinforced concrete groynes had been rather unfortunate; it tended to show that although these might be extremely useful in some cases, one needed to be very careful that the conditions were really suitable for their use. He thought probably they would be very good on sandy beaches or on rocky beaches; but where you had a shingle beach, the attrition of the shingle very rapidly wore away the concrete and exposed the steel structure. In several cases that he had seen lately, the steel had been largely destroyed. He moved a very hearty vote of thanks to Mr. Noble Twelvetrees.

Mr. S. Bylander seconded the vote of thanks. He said they ought to be proud of the early pioneers who, with confidence and enthusiasm, had carried on a very definite work.

Mr. Twelvetrees had emphasised the fact that reinforced concrete was a wonderful material, and that very little damage would result to a structure where it was used, from collisions or bad treatment. Anyone who had tried to steer a ship and place it next a jetty would realise the enormous stress there was, and the bending of the piles or piers in the jetty. He remembered once trying to steer a ship near to a jetty, which was of timber, and he thought the jetty must have moved very nearly a foot. On the point of the need for stability and strength, Mr. Twelvetrees had said in the early part of his paper how difficult it was to get braces below water level, and he (Mr. Bylander) would explain a method that he had recently seen employed for a bridge where there was very great depth of water, and where, for reasons of cost, it was not desirable to use caissons. Piles over a hundred feet long were made about two feet six inches in diameter and hollow. These piles were made with exceptional care to make sure that the steel was properly protected. Concrete about six inches in thickness was used. The piles were made on the shore, and the upper end closed temporarily. The pile was then floated into position, raised by cranes placed on barges, lifted into the vertical position, and sunk by means of a water jet through a pipe inside the pile. In that manner the piles were sunk to a great distance into the sand, and after being placed in position at a satisfactory depth another pipe was inserted into the hollow portion of the pile and the inside filled with cement grout, so that finally there was a solid concrete pile capable of resisting horizontal pressure as well as carrying the load.

Mr. W. J. H. Leverton said that with regard to what had been said about reinforced concrete being suitable for groynes, if these were cast *in situ* they seemed to be quite useless unless they were protected from the waves for a very long time, which was impracticable. He had been reminded of what he had seen at Shoreham when Mr. Twelvetrees had put on the screen an illustration of a pre-cast groyne. He supposed this was the result of experience.

Mr. Colson said with regard to bringing ships alongside concrete jetties, he had had to deal with torpedo boat destroyers; these might be likened to very thin egg-shells. The difficulty had been got over by building a jetty on reinforced concrete piles, but with the whole of the upper work of timber. When a destroyer came alongside the jetty gave slightly and did not damage the ship. Another point with regard to reinforced concrete jetties was that it was necessary to look out for corrosion not only in the piles but

on the side of the deck that was above the water altogether. Although it would be expected that the beams which supported that deck would be quite dry, he fancied the water condensed on them. There were frequently cracks along the centre of those beams.

Mr. de Vesian said it took one's memory back many years to see some of the older works illustrated on the screen. He had a feeling of envy for Mr. Hennebique for the boldness of the works and designs put forward at that time. Every such work must have been a surprise to engineers; in fact, reinforced concrete generally, when it first came forward, was a series of surprises to the older generation. The idea of a man having the impudence to suggest moulding a stick of concrete sixty feet long and 14 inches square, and driving it into the ground with a heavy hammer! To most engineers at the time it was absurd. But it had been done. He thought it was greatly due to Mr. Hennebique's boldness, backed by scientific and technical knowledge, that we owed so much advance in reinforced concrete construction. Perhaps the pile had been one of the most outstanding surprises. Lately at Newcastle he had had piles being driven with over *twenty-four thousand blows* on one pile to get it down. As to groynes, about a month ago he had seen some at Rottingdean which had been made twelve years ago; and had been surprised to find them in so perfect a condition. The shingle which had been collected consisted, in some cases, of rounded lumps of flint about as big as a cricket ball. That shingle had been driven up and down those groynes but they remained perfect after twelve years.

The Chairman: Mr. de Vesian has spoken of some piles which have stood twenty-four thousand blows. It would be interesting to know the size of the hammer used.

Mr. de Vesian: They were fourteen by fourteen piles. It was a two-ton hammer, and, I believe, three feet six inch drops. It was being carried out for the City Engineer.

Mr. White said he would like to know how far the sea water percolated through the concrete in the reinforced concrete piles, and whether there was any rendering done to prevent such percolation.

Professor Matthews: Is it a fact that owing to reinforced concrete piles often being bound to absorb the sea water into which they are driven that engineers are now experimenting with all sorts of solutions to coat the piles before they are driven, and that one of the materials they have used for that purpose is tar.

A certain dock engineer told me that he had a trough of tar made, picked the pile up by a crane, dipped it into the tar and then drove it. He said he had broken open one of the piles treated in that way several weeks after, and found that the tar was doing its work well by preventing the absorption of the sea water.

Mr. Gardner said that Mr. Twelvetrees had made special reference to the bow-string bridges, one example being sixty feet and the other one hundred feet span. That type of bridge seemed to be most suited for the reinforced concrete construction, being virtually a tied arch, with not much work coming upon the web system. But one would be interested to know for what weights, rolling loads, &c., the bridges were designed. With regard to the protection of the reinforcement, it seemed to him there was a difficulty. If you put on a thicker protection to the under side of beams and decking, you would get bigger hair cracks as canals for moisture.

Mr. Crocker said he would like to put in a plea for the simplification of design in reinforced concrete jetties and marine works at the present time, in view of the inflated price of Portland Cement and high wages. No doubt considerable time was lost in the construction of jetties in tidal works, and the fixing of shutters and a complicated arrangement of piers entailed considerable delay, labour and cost. He did not know how this idea was to be carried out; possibly it could be done by adopting more extensively construction by means of casings and in cylinders.

Mr. Noble Twelvetrees said he thought it would be best to reply to the discussion in the *Journal*. He thanked the mover and seconder of the vote of thanks, and the meeting for passing it.

MEETING IN MARCH.

The 110th ordinary general meeting will take place at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, March 23, 1922, when Mr. S. F. Staples, M.Inst.C.E., M.I.N.A., will read a paper (illustrated by lantern slides) on "Floating Docks."

COUNCIL APPOINTMENTS.

The Council has elected Mr. H. J. Deane, M.Inst.C.E., a Vice-President, and to fill the vacancy on the Council so caused they have co-opted Professor E. R. Matthews, A.M.Inst.C.E.

PROCEEDINGS.

The 107th ordinary general meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, December 15, 1921.

The chair was taken by the President, and the minutes of the previous meeting, as published in the *Journal* were taken as read, and were confirmed and signed.

The following members were then elected in accordance with Article 8 of the Articles of Association :—

Members.

Bayliss, Francis Charles, F.S.Arc., "Keith," Fitzalan Road, Finchley, N.3; Burton, Duncan McLean, F.S.Arc., P.O. Box 3565, Johannesburg; Chapman, James, 46, King's Road, Sloane Square, S.W.3; Claessen, William, F.S.Arc., Bristol Buildings, Fort, Colombo; Lonnon, Ernest, 35, Wrottesley Road, Harlesden, N.W.; Gardner, George Anthony, L.C.C., New County Hall, S.E.1; Knipe, Adam Cyril, 12, Rochester Row, S.W.1; Nimmo, James Valence, M.Inst.C.E., Messrs. Monod & Guillain, 72-74, Victoria Street, S.W.1; Roberts, George Vincent, County Surveyor's Office, High Town, Hereford; Weaser, Lawrence Thomas, Old Court, Chelmsford; Wulff, Johan Christen, Messrs. Christiani & Nielsen, 72, Victoria Street, S.W.1; Dennis, Nelson F., M.Inst.C.E., "Greenside," Foggy Furze, West Hartlepool.

Associate Members.

Hyland, Henry, M.S.A., Messrs. Hyland & Phillips, Hastings H.B., New Zealand; Pearson, Edward M., Harris Institute, Preston; * Turner, Ernest Charles, Seaton House, Harrow Road, N.W.

**Passed Examination.*

In view of his approaching departure for South Africa, Mr. E. J. Hamlin, D.Sc., A.M.Inst.C.E. (Associate Member) took precedence of the lecturer for the evening, and delivered a short paper on "The Effect of Humidity, Temperature, and Water Content on the Strength of Concrete."

A full discussion of the paper was postponed pending its publication, and a hearty vote of thanks was accorded to Mr. Hamlin.

Mr. W. Noble Twelvetrees then read a paper (reported in this issue) on "Reinforced Concrete Piers and Marine Works." A lengthy discussion followed, and the lecturer was accorded a hearty vote of thanks, to which he responded.

The proceedings then terminated.

A CONCRETE INSTITUTE MASONIC LODGE.

Some members of the Concrete Institute are desirous of founding a Masonic Lodge, associated with the Institute. Members of the Institute interested are invited to communicate with the Secretary of the Institute, Captain M. G. Kiddy, 296, Vauxhall Bridge Road, S.W.1.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	{ First 500 ...	£1	1s. 0d.
		{ Remainder ...	£2	2s. 0d.
ASSOCIATES	,, ...		£2	2s. 0d.
ASSOCIATE-MEMBERS	,, ...		£1	11s. 6d.
GRADUATES } STUDENTS }	,, ...			10s. 6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

REVIEW.

"The Designers of our Buildings." By L. Cope Cornford. Published by R.I.B.A. London, 5s.

Mr. William J. Locke, the novelist, who was at one time Secretary to the Royal Institute of British Architects, writes a notable "foreword" to this book, in which he says, "if you asked any sixty men at an ordinary London club to write down the names of six distinguished architects, not more than two or three could do it." He adds. "The fault, to some extent lies with the architects, they deprecate advertisement."

This is true, but none the less, this book may be described as a neat advertisement of architects in general, and of the Royal Institute in particular. It is a book which is written to excite curiosity (in the best sense of the word) about the Institute and its work.

Mr. Cornford is no apologist for the much abused nineteenth century, and thinks that its architecture was largely due to its steady demand for what was cheap and ugly. During this dark age he says "the architects held to their art, and by virtue of the patronage of the intelligent few, even contrived to live by it."

In 1835 the Royal Institute was founded, and since then it has grown in numbers and influence. It now has an income of £19,000 per annum, and administers trust funds of the value of £30,000. It is consulted by Government Departments and public bodies. Practically every architect worthy of the name is numbered in its ranks, and all have to pass the examinations, and tests, which it imposes. It suggests the lines of the architect's education and training, gives prizes and scholarships to encourage him, provides a magnificent library for his assistance, and a benevolent fund for his misfortunes. It lays down procedure for inviting competitive designs for public buildings, and insists that the prizes for such competitions shall be awarded only by a competent architect.

Its code of professional conduct and practice is an admirable mingling of discipline and inspiration. It reminds the architect that he is at once an artist, a man of science, an economist, and a trustee. His aim must be to achieve perfection in all these lines.

The book is illustrated with portraits of famous architects of past time. It is published by the Institute, and forms a record of their work, of which they and the author may well be proud.—F.E.W.S.

NORTHAMPTON BOOT FACTORY FIRE.—Five hundred men were thrown out of work by a fire at Messrs. Barratt's "Footshape" boot factory at Northampton yesterday. A concrete floor prevented the fire from spreading to the huge stock rooms containing more than 80,000 pairs of boots and a card index system relating to over 120,000 customers.—*Daily Mail*, January 25, 1922.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *MARCH, 1922.* **No. 3.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

THE BUILDING EXHIBITION, 1922.

By the courtesy and kindness of Mr. H. Greville Montgomery, director of the Building Exhibition, the Council has been able to arrange for the holding of a series of discussions on " Problems of the Moment," full particulars of which will be found in the pamphlet accompanying this number of the " Journal."

Special arrangements have been made for a furnished sitting room to be placed at the disposal of members and their friends visiting the Exhibition.

The sitting room will be situate over the main entrance to the Exhibition.

The Secretary of the Institute will be at the Exhibition daily after 3 p.m., and will be pleased to meet any members of the Institute visiting the Exhibition.

Early application should be made for tickets for the discussions, and also for tickets for the fancy dress ball and carnival organised by the Architectural Association, which is being held at the Exhibition on Friday, April 21. The tickets for the dance are £1 1s. single tickets, and £1 15s. double tickets, including buffet refreshments.

Tickets for all functions may be obtained from the Secretary of the Concrete Institute.

CAPILLARY CANALS IN CONCRETE AND THE PERCOLATION OF WATER THROUGH THEM.

BY E. B. MOULLIN, M.A.

ORIGIN AND SCOPE OF THIS PAPER.

The experiments which this paper attempts to describe were first suggested by reading the report on Ferro Concrete issued by the Institution of Civil Engineers, which describes many experiments made in 1911 at the National Physical Laboratory on the percolation of water through concrete slabs.

The tests described in this paper were made over three years ago under very disadvantageous conditions in regard to available time and facilities for research. They are in a way a repetition of the National Physical Laboratory's experiments on a very small scale.

Attempts have also been made to examine the internal structure of hydrated cement and plaster by means of microscopic examination of thin sections ; some photographs so obtained are shown reproduced. These photographs give some general information as to the size and types of the capillary canals and cavities formed in the hydration of cement and plaster, and may possibly be helpful in elucidating the physical conditions of setting.

A simple theory of the laws and conditions of absorption has been formulated, and its mathematical interpretation is found to be in very close agreement with experiments.

The author feels afraid that this paper is more of scientific than practical interest, and fears that it has little direct bearing on the art of Concrete Construction. However, there is always the hope that any knowledge will ultimately find some useful and practical application, and may this be his apology and excuse.

Description and analysis of the experiments made at the National Physical Laboratory in 1911.

These experiments showed the rather surprising fact that water does not pour through concrete at a constant rate. When the slab is first subjected to pressure, the water passes through fairly freely, but in the course of some thousand hours or so the rate of flow diminishes to about one-thirtieth part of the initial value.

An explanation of this time reduction of percolation was that the slabs became silted up with calcium carbonate, either present initially in the water or eroded from the interior of the slab by the passage of the water. However, the same slabs were after a rest again subjected to a reduced water pressure, and again showed the same phenomenon of diminishing rate of flow. Fig. 1 is an example of one of several curves given in the report, and serves to show this point.

If silting up is the explanation, then, presumably, water should always pass at a constant rate through a slab that has once been silted up by previous exposure to water, but experiment shows that this is not the case.

Percolation and Absorption Experiments Made by the Author.

The following experiments were all made on a very small scale, and Fig. 2 shows diagramatically the apparatus used.

The quantity of water absorbed was measured by the movement of the water meniscus in the horizontal portion of the capillary tube; the readings were carefully corrected for temperature changes by means of a correction curve for the apparatus obtained experimentally.

The test blocks were cylinders about 6 cms. diameter and 6 cms. high. Fig. 2b shows the modification adopted to measure percolation through a block as well as absorption into it.

Curves Obtained from these Experiments.

Fig. 3 shows an absorption curve for a Plaster of Paris block, and Fig. 4 for a specimen composed of cement and very fine gravel.

These curves show a decreasing rate of absorption similar to the decreasing rate of percolation found in the National Physical Laboratory Tests.

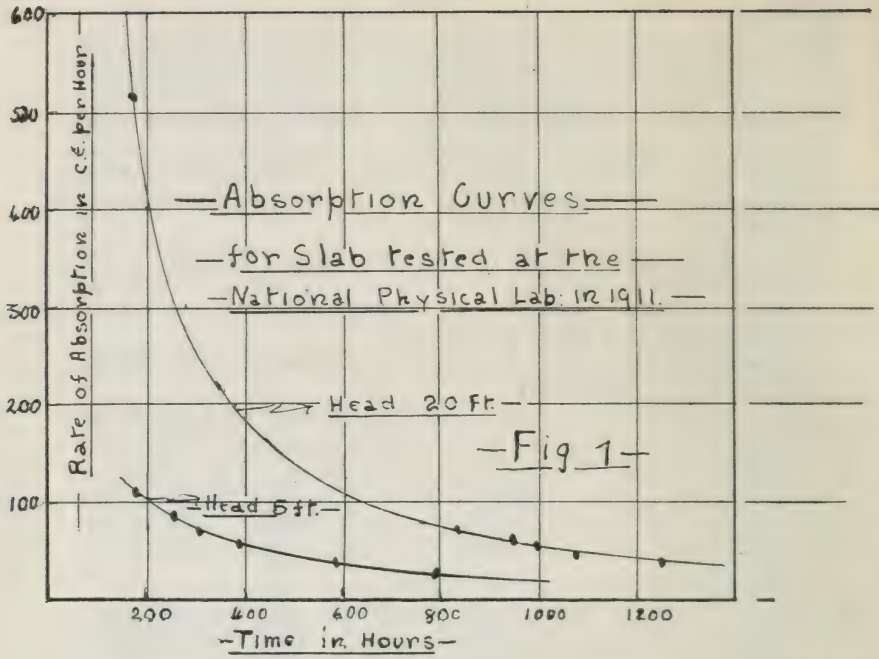
Fig. 5 is another absorption curve for a cement block, and 5a is the curve obtained from the same block when the test was repeated 13 days later, showing that it had not become silted up in the first soaking.

Fig. 6 is the curve obtained for the case of water flowing into one side only of a block, the other sides being open to the atmosphere.

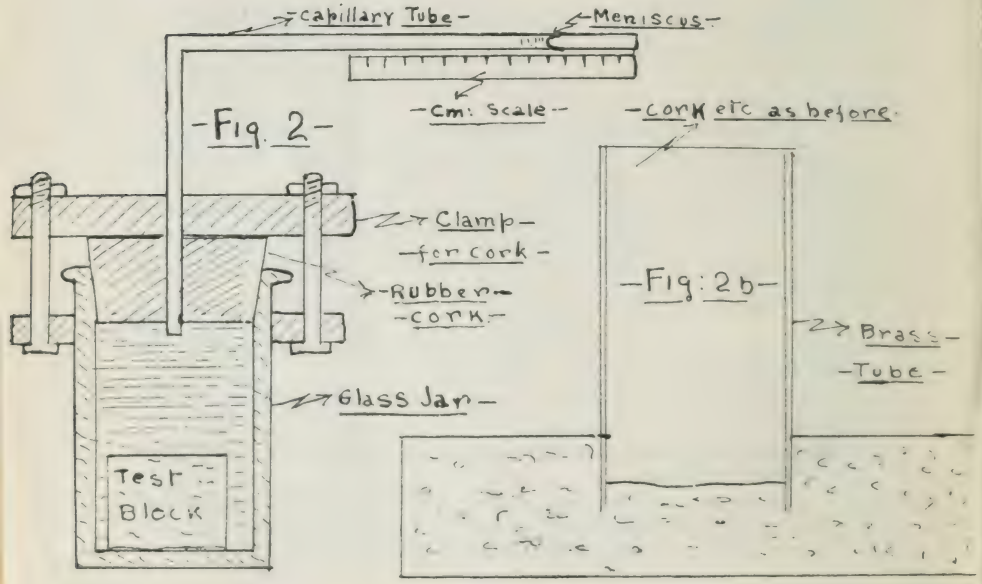
The straight lines drawn on all these curves will be referred to and explained subsequently.

Nature of Internal Structure as Revealed by the Microscope.

Microscope slides were made consisting of thin sections of cement and plaster, thin enough to transmit light through them; this is a method of attack familiar to petrologists.



-Fig 1-



-Fig. 2-

-Fig: 2b-

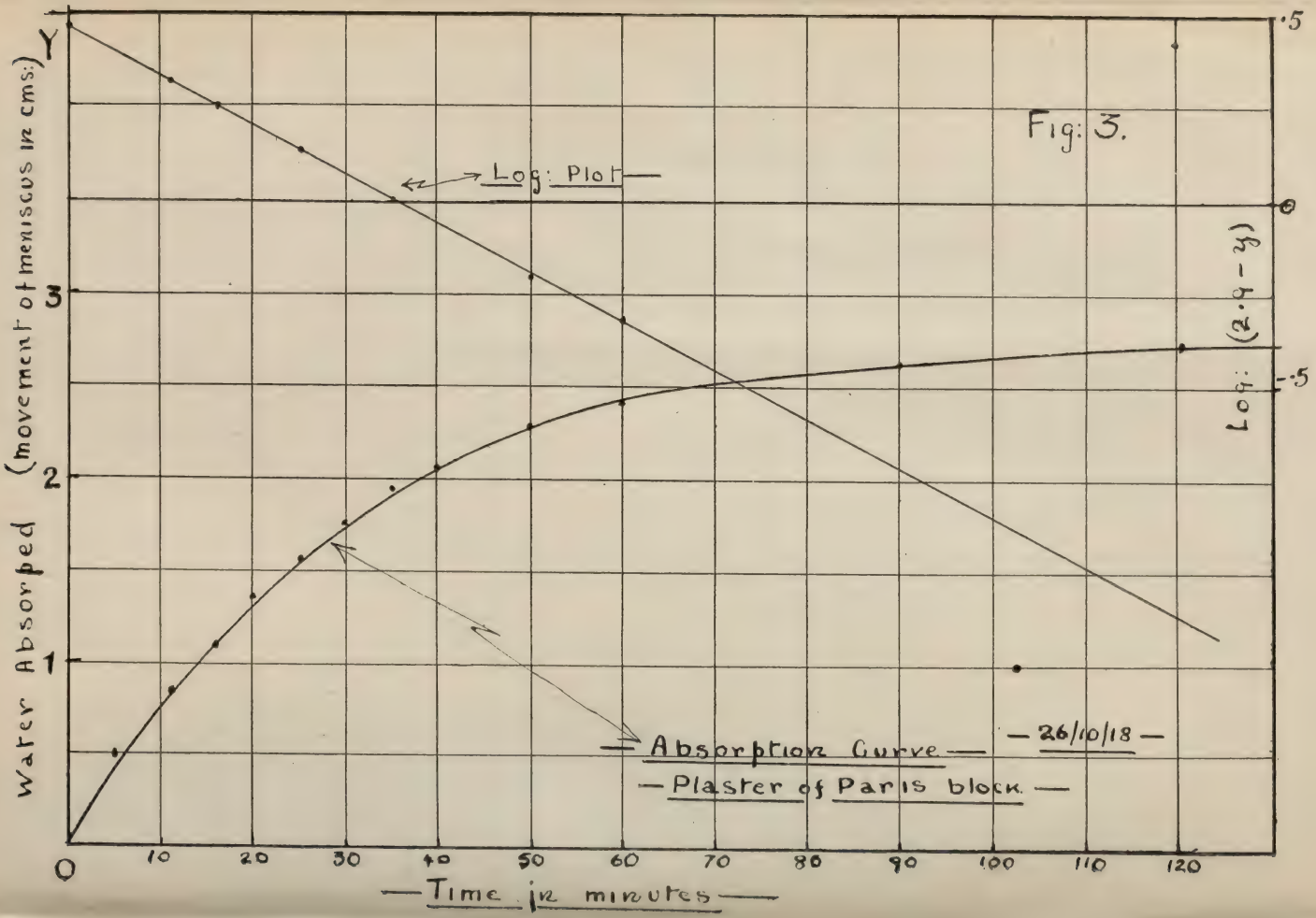


Fig: 3.
Water Absorbed (movement of meniscus in cms:)
Y
3
2
1
O
Log: Plot
Log: (2·4 − y)
·5
0
·5
10
20
30
40
50
60
70
80
90
100
110
120
Time in minutes
Absorption Curve
Plaster of Paris block.
26/10/18

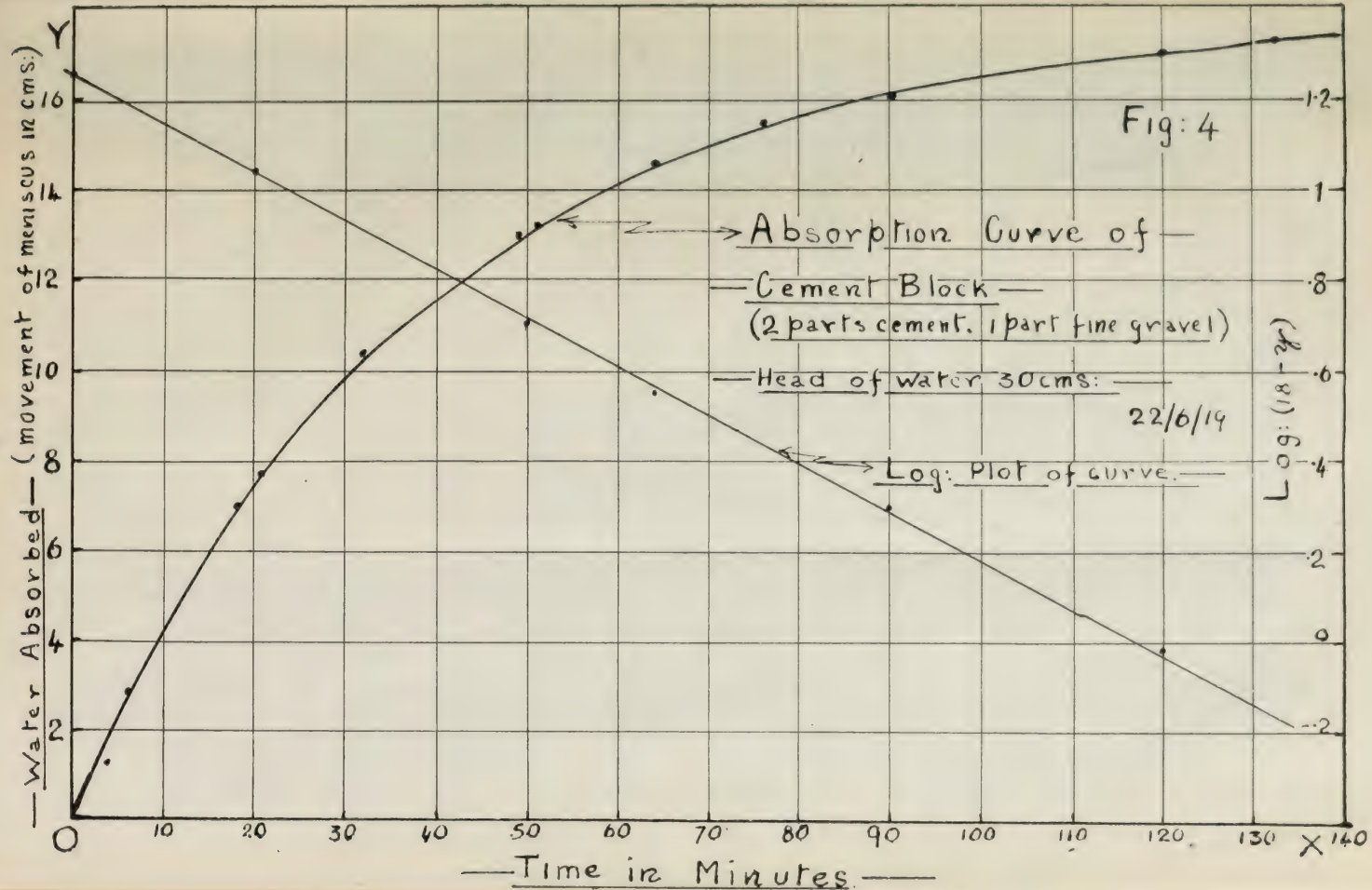


Fig: 4
Absorption Curve of
Cement Block
(2 parts cement. 1 part fine gravel)
Head of water 30cms:
22/6/19
Log: Plot of curve.
—Water Absorbed— (movement of meniscus in cms:)
Y
16
14
12
10
8
6
4
2
O
10
20
30
40
50
60
70
80
90
100
110
120
130
X 140
—Time in Minutes—
1·2
1
·8
·6
·4
·2
0
-·2
Log: (18 - 2r)

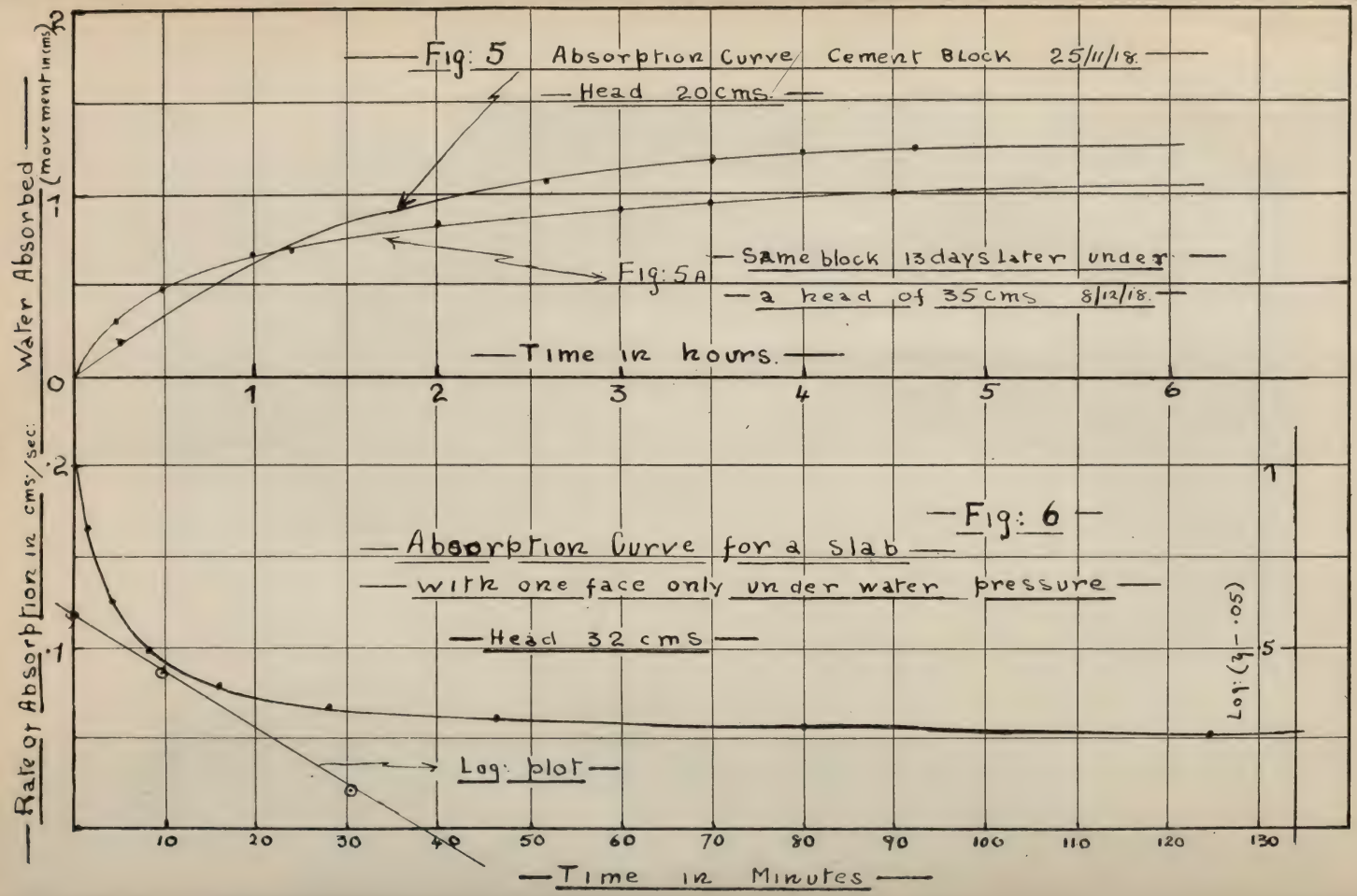


Fig: 5 Absorption Curve Cement Block 25/11/18.
Head 20 cms.
Fig: 5A
Same block 13 days later under a head of 35 cms 8/12/18.
Time in hours.
Water Absorbed (movement in cms)
0
1
2
3
4
5
6
Fig: 6
Absorption Curve for a slab
with one face only under water pressure
Head 32 cms
Log: plot
Rate of Absorption in cms/sec.
Log: (y - .05)
10
20
30
40
50
60
70
80
90
100
110
120
130
Time in Minutes

Examination of cement in this manner clearly shows the remains of partly dissolved cement grains: in some cases the original internal crystalline structure of these grains is visible.

The intervening spaces between the grains is filled with a dense amorphous substance, which is presumably the result of the desiccation of the colloidal solution formed by the mixture of cement and water. Piercing through this amorphous substance are a large number of circular holes, which must be cross sections of spherical cavities in the block. In one slide there are 55 of these holes in an area of about two square cms. of cement.

Fig. 6A shows one of these holes highly magnified, and its wonderfully circular shape is remarkable.

The light substances near the hole are sections of small quartz pebbles mixed with the cement; the remainder of the slide is set cement, and the darker flecks in it are mostly partly dissolved cement grains.

The diameter of these holes varies from seven-tenths of a millimetre to a little under one-tenth of a millimetre, but as these holes are not necessarily diametral planes of the spherical cavities their diameter does not show the average size of the spherical hole. Probably .7 mm. may be taken as an upper limit.

If a block of cement concrete is cut through and its surface ground dead flat, these cavities are easily seen, some being greater and some less than a hemisphere.

The insides of these cavities can be observed with the help of a powerful microscope, and viewed in this manner they present a remarkably beautiful appearance. The walls are completely covered with tiny crystals of varying hues of green, blue and grey, and in the absence of anything to give true scale to the picture they might be mistaken for a gem-studded grotto.

Owing to the difficulties of obtaining adequate illumination, and the trouble caused by parts necessarily being out of focus, it is very hard to obtain a satisfactory photograph of these interiors.

Fig. 7 shows an attempt at such a photograph, and the crystal-studded walls and bottom can be seen fairly well. The dark crescent on the left is part of the wall in deep shadow owing to the light not being perfectly central. The actual size of the cavity shown here is about that of the head of a small pin.

The depth of these cavities can be measured by noting the movement necessary to focus first on the top and then on the

bottom of the holes. By measuring both the depth and the diameter of the cavity the volume of the sphere of which it formed a part is easily calculated. In the case of eight holes thus measured the radii of the spheres lay between one-tenth and five-tenths of a millimetre, giving volumes of 4.2×10 cub. cms. and 525×10^6 cub. cms. respectively.

Thus the largest measured had 125 times the volume of the smallest, so there seems no tendency for the cavities to form equal in size. Exactly how these caverns are formed is not clear, but from their spherical shape they are presumably caused by drops of excess water. These slides seem to suggest that more cavities are formed close to a pebble of aggregate than not, but as they also appear in great abundance in Plaster of Paris, the presence of pebbles is evidently not essential to their formation: neat cement blocks as a rule do not show many cavities.

It would be probably worth while to see if the number of holes per square c.m. is related to the percentage of water used in mixing, and also their effect on the tensile strength of a block.

No doubt these cavities are similar to those observed by Mr N. C. Johnson, and referred to in Mr. Andrews' paper on "Some methods of securing impermeability in concrete."

Apparently the "Michaelis" theory of the setting of cement claims that there is a deposition of hydrated tricalcium aluminate from a super saturated solution: and that this deposition takes place in a partly colloidal and partly crystalline form: an excess of water appearing to favour the formation of crystals.

In these slides the bulk of the cement appears to be amorphous, but the walls of the cavities are very crystalline: if these cavities are formed by excess moisture this theory is apparently supported: in any case there is apparently a discontinuity of structure near the surface of internal cavities.

Presence of Capillary Canals Revealed by the Microscope.

Besides spherical cavities the microscope also shows a certain number of minute canals following tortuous paths through the cement.

Fig. 8 shows a hole with two of the canals entering it; the width of these ducts in between one and three-thousandths of a millimetre, or, roughly, forty-millionths of an inch.

The slides show several canals in the cement, but only one case of a duct entering a cavity; this is not surprising when the

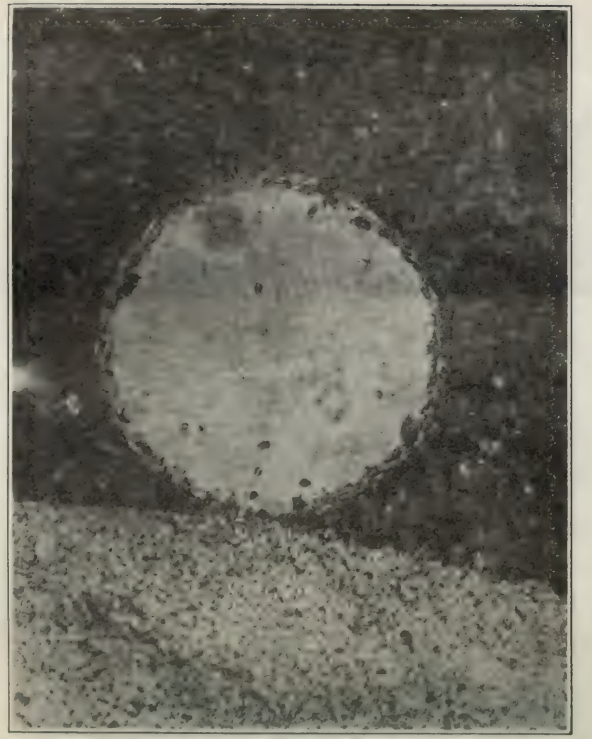


Fig. 6a. Magnified 44 diams.

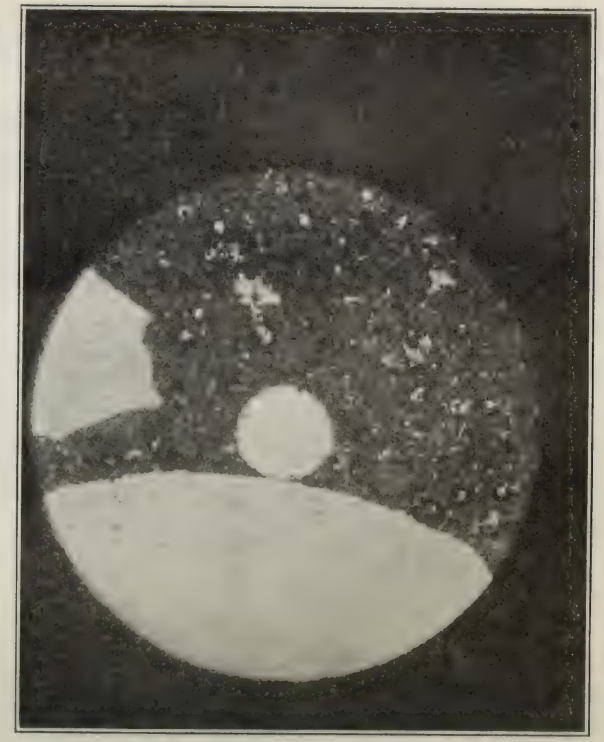


Fig. 6b. Magnified 140 diams.

Fig. 8.—150 Diams,, Approx.

Fig. 7.

minute size of these canals is remembered. For the ducts being very small, even compared to the cavity, the chances of the section being cut at the exact spot where a duct enters the cavity are very small. Hence, although the microscope reveals the existence of these ducts, and proves that ducts do enter cavities, it cannot be expected to prove that all cavities have canals leading to them, although it is highly probable that this is actually the case.

Some indirect evidence on this point is given by taking a block that has one surface ground dead smooth, and smearing this surface copiously with water. If the surface is viewed through the microscope it is seen that the water surface instantly goes hollow and the film quickly breaks over each spot where there is a cavity. Also, if a drop of water is placed in a cavity it quickly runs into the block, and the receding hollow surface can be watched like water running out of a small tank; a drop placed on the smooth surface of the block, not in a cavity, takes a much longer time to disappear.

Whether or not the mass of the cement is permeable apart from these canals it is impossible to say, but it seems probable that the bulk of the percolation takes place through these ducts, and the cavities form frequent tiny reservoirs in their course.

In fact, the internal structure of the cement may be compared to an extensive system of tube railways, with frequent stations where the size of the tube is greatly enlarged.

A Theory to Explain why Absorption takes Place at a Decreasing Rate.

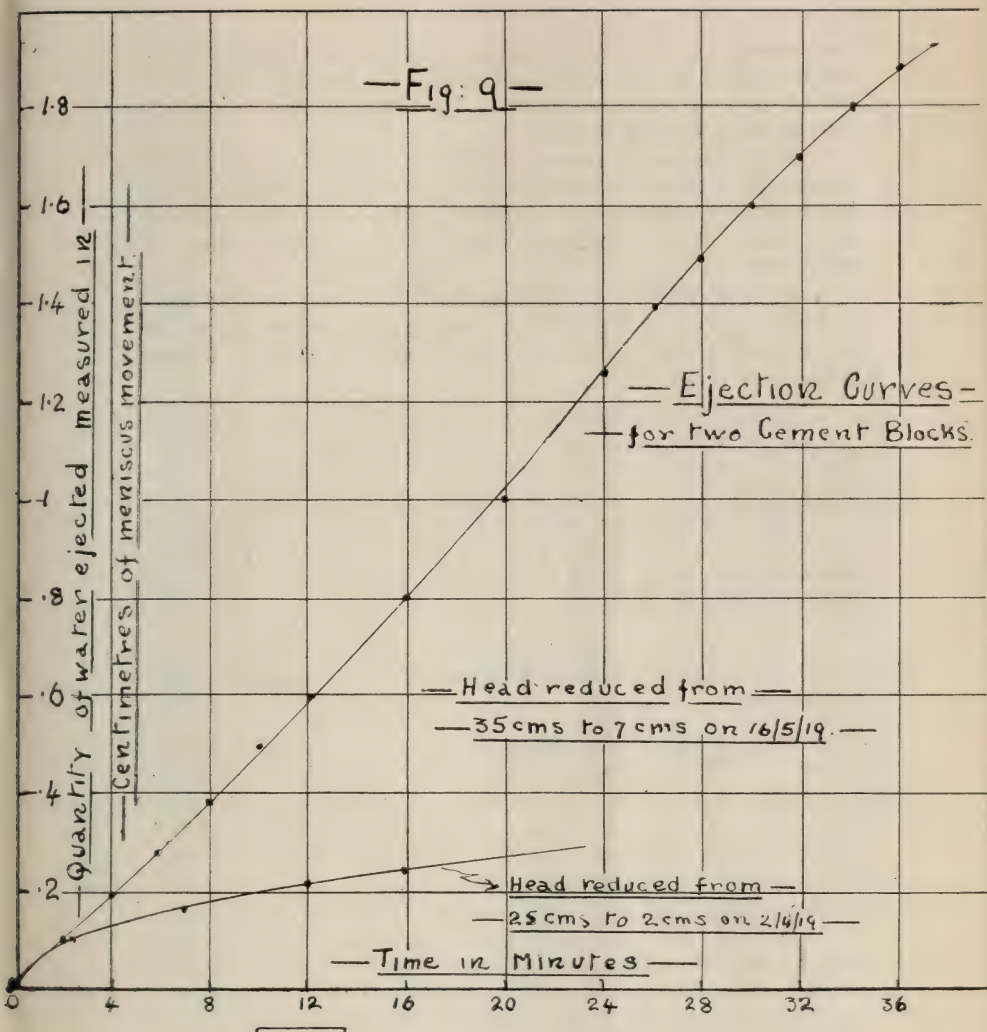
The passages and cavities in the cement are initially filled with air under atmospheric pressure, which has to be displaced wholly or partly by the entering water.

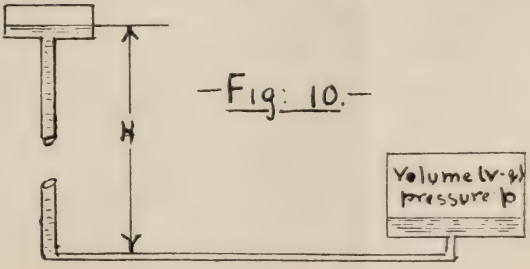
Consider, firstly, the case of a block completely submerged in water: The air initially in the block is completely trapped, and, consequently, must be driven by the entering water, through the passages and caverns towards the centre of the block.

Its original volume is thus reduced, causing a continuous rise in internal pressure as the water enters.

The rise of internal air pressure reduces the resultant force causing the inflow of water, and as this resultant force will decrease continuously as more water enters, the rate of absorption must be a decreasing function of the time, which experiment has shown to be the case.

The suggested sequence of events may be compared to a small scale reproduction of what takes place when a sea wave


—Fig: 9—
1·8
1·6
1·4
1·2
1
·8
·6
·4
·2
Quantity of water ejected measured in Centimetres of meniscus movement.
—Ejection Curves—
—for two Cement Blocks.
—Head reduced from—
—35cms to 7cms on 16/5/19.—
Head reduced from—
—25cms to 2cms on 2/4/19—
—Time in Minutes—
0
4
8
12
16
20
24
28
32
36


H
Volume V at pressure p

—Fig: 10.—

rushes into a cavern that is nearly submerged ; the imprisoned air, becoming highly compressed, resists the entry of the wave. When the wave recedes it helps to eject it, and ultimately bursts out in a cloud of spray.

If the decreasing rate of absorption of the water is due, as suggested, to the compressing of the air that is initially in the block, then it should be possible to make this compressed air eject some of the absorbed water if the external head is removed.

Experiments to test this showed that such was the case. and the curves of Fig. 9 show how water exuded when the external head was removed : in both cases the block had been submerged many hours.

It is difficult to exactly foretell how surface tension forces will act in such a case, but it is probable that on the whole they will assist absorption and resist ejection, so that it is to be expected that the quantity ejected would be small.

However, the fact remains that ejection does take place when the head is removed, and it is extremely hard to see how this can take place except under the action of air compressed in the cavities of the block.

Mathematical Interpretation of Trapped Air Theory.

As a first approximation consider the case of a single closed vessel being filled through a very small pipe, the water being under a constant head "h." Let the volume of air initially in the tank be "V" at atmospheric pressure "P." After time "t" when a quantity of water "q" has entered let the pressure in the vessel be "p."

Then, supposing no change of temperature to have taken place $p(V-q)=P.V$. Neglecting surface tension forces, it is easy to show that the total quantity "Q" which will enter is given by the equation $Q(h+P)=hV$; now in these experiments "h" is about $\frac{1}{30}$th of "P," so that, finally, $Q=\frac{1}{31}V$: hence, as the greatest value of $\frac{q}{V}=\frac{1}{31}$, therefore $(\frac{q}{V})^2$, &c., can be neglected in comparison with $(\frac{q}{V})$.

The resistance to flow through the pipe will vary as the speed of flow, which is equal to $\frac{1}{\pi k^2}\frac{dq}{dt}$, and hence the resistance to flow is given by $k\frac{dq}{dt}$ where k is a constant depending on the viscosity of water and the length and radius of the pipe.

Hence the equation of motion becomes,

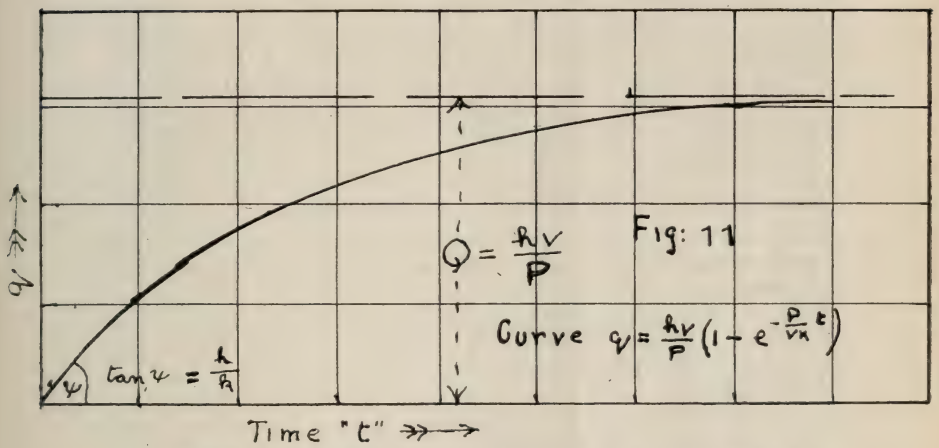
$$P + h - p = k\frac{dq}{dt}$$

$$\therefore P + h - \frac{PV}{V-q} = k\,\frac{dq}{dt}$$

$$\therefore (P + h) - P\left(1 - \frac{q}{v}\right)^{-1} = k\,\frac{dq}{dt}$$

$$\therefore h - \frac{Pq}{v} = k\,\frac{dq}{dt} \text{ neglecting } \left(\frac{q}{v}\right), \&c.$$

and hence $q = \frac{hV}{P}\left(1 - e^{-\frac{P}{Vk}t}\right)$



Which is a simple exponential curve, as shown in Fig. 11.

This curve is of the same general nature as these found experimentally, but as it only refers to one cavity it cannot be applied without further consideration.

It is quite unsuitable to consider the mathematics of an infinite distribution of similar cavities and passages one after the other, but a fair insight into the conditions obtaining in an actual case can be got by considering the case of two and of three tanks in succession.

The result of such a calculation is as follows, for two tanks :—

$$q = 2\,Q\,(1 - \cdot 9\,e^{-m} - \cdot 05e^{-7m})\text{ where } m = \cdot 4\frac{P}{kV}$$

and for three tanks is :—

$$q = 3\,Q\,(1 - \cdot 9\,e^{-n} - 0 \cdot 8e^{-3n} - \cdot 01e^{-17n})\text{ where } n = \cdot 2\frac{P}{kV}$$

This shows that the terms of the series representing a large number of tanks decrease in value very rapidly, and except when " t " is very small the first term of the series is the significant one.

Consequently there is reason to suppose that the quantity of water entering a block can be represented by an equation of the form $q = Q\ (1 - e^{-xt})$ where "x" is some constant depending on the number and size of the cavities and canals.

It is, fortunately, very easy to test whether a curve is of this form, for if it is it will be found that when the logarithms of the ordinates are plotted against the corresponding values of the time (not the logarithm of the time) the result will be a straight line. This has been done in all the experimental curves already shown, and it is seen that the result is a straight line to a wonderful degree of accuracy. Hence that these soaking curves are true exponentials seems to be established, and as this is the curve required by the supposition that the controlling factor in the soaking is the air imprisoned in the block, there seems good reason to believe that the simple explanation suggested is correct.

As the air in the cavities is under pressure, it will presumably enter into solution in the water and slowly diffuse through the canals since the concentration at the surface is less: the time required to do this would be very long compared with the time required to complete the processes we have been considering. However, the result would finally be that the imprisoned air would escape, and that after a very long time every cavity and canal in the block would be completely filled.

When comparing the porosity of stones or bricks by weighing them after immersion in water, it is apparently important to soak the specimens for many days in order to obtain a true comparison—unless the period of immersion is very long the imprisoned air will not have escaped, and the specimens need not necessarily have attained the same degree of possible saturation.

It is, perhaps, not without interest to substitute numbers in the expression already found and find how long it would take to fill one cavity of the size known to exist in the concrete through one single canal (supposed circular) of the size measured.

Suppose the length of a duct is 1 mm. and its radius $\frac{1}{10^4}$ cms.

$$k = \frac{8c}{100\pi R^4} = \frac{10^{12}}{6} \text{ about.}$$

Take $V = \frac{1}{10^4}$ cub. cms. and $P = 1{,}000$ grams. per sq. cm.

$$\frac{kV}{P} = \frac{10^{12}}{6} \times \frac{1}{10^7} = \frac{10^5}{6}$$

When $t = \frac{3vk}{P}$ the final state will be nearly reached.

Time required $= \frac{10^5}{2}$ seconds.

$= 13$ hours about.

No doubt in most cases there are several canals per cavity, and this would reduce the time required; in any case, the time calculated above is of the right order of magnitude.

If the analysis is performed for the case where the applied head is comparable to the atmosphere so that $\left(\frac{q}{V}\right)^2$ cannot be neglected, the curve is no longer a simple exponential, though it is of the same general type: it, however, only differs appreciably in the early stages.

Consideration of Absorption into and Percolation through a Slab Exposed on One Surface Only to Water Pressure.

Absorption at a decreasing rate into such a slab can be explained on similar grounds, for all the central portions will be surrounded with water and behave in a similar manner to a submerged block.

Also there will be considerable resistance to flow of air in the direction of the thickness of the slab which will all tend to act in the same manner and allow time for the water to enter to varying depths and completely enclose a region. Fig. 6 is an example of a curve obtained for such a case: the rate of flow is still fairly well exponential, though not so accurately as for the pure absorption case.

The Physical Laboratories tests showed that water actually passed through and dripped from the underside at a decreasing rate: it is difficult to see how this can be accounted for in the same way, although perhaps with more knowledge of the distribution of the canals it would be clear.

It will be noticed that after 1,800 hours the rate of outflow is still only about half the rate of inflow, showing that absorption is still taking place. The difference between the inflow and drip curve will give the rate of absorption, and this is fairly closely exponential. However, it still seems necessary in these cases to look for some contraction of the passages, although it has been seen that this contraction cannot be permanent.

If such contractions do take place there is no need to consider them in the absorption tests considered above, for the length of time there considered is very small compared to the length of tests at the Physical Laboratory.

It would be interesting from many points of view to discover whether the presence of water in the canals causes the small grains to swell and obstruct the passages, contraction taking place later when the substance had partly dried off.

It is generally accepted that permeability decreases with age, and it would be interesting to know whether this ageing can be accelerated by the passage of water for long periods.

It is difficult to see how the microscope can be of assistance in tracing progressive internal changes in cavities and canals, although it may, perhaps, assist expert chemists to tell us how to make cements and mortars initially free of these defects; but until this happy discovery has been made, the evidence of these photographs seems to indicate that some method of silting up the concrete might be as effective as adding various compounds with the cement before mixing.

Discussion.

The PRESIDENT, after paying a tribute to the author for having given such an excellent paper, read a letter on the subject of the paper from Dr. Oscar Faber, quoted hereafter.

Dr. J. S. OWENS proposed a hearty vote of thanks to the author for his paper, particularly as the subject of the micro-structure of cement and concrete was far too little ventilated. The paper had the advantage that it opened up what was, to an extent, a new aspect of the question of the structure. There were some points in the paper with which he agreed. The method of testing the absorption of water in concrete, in which a small cube is enclosed in an airtight and watertight vessel, cannot result in anything but the result shown, i.e., they had a volume of air imprisoned inside the concrete, and the water as it went in had to compress that air. Consequently, the curve showing the degree of compression of the air was found to be of the nature shown. The question of the absorption of water in the block was, therefore, in that case mixed up with that of the com-

pression of the imprisoned air. It was worth remarking that there was no means of escape of that air. In the case of the percolation of water through a slab, he did not quite agree with the author's conclusions that the diminishing rate of percolation was also due to imprisoned air. He (Dr. Owens) could not quite see that the statement that the air was under similar conditions to that in the block surrounded by water was correct. The slab was not surrounded by water on all sides, and the air might escape through the lower face, so that he thought they must look for some other cause for the diminished rate of flow, and the obvious cause, he believed, was the contraction of the canals. He used the word " canals " in a wider sense, i.e., there must be passages of some sort, otherwise water could not get in. Perhaps those passages contracted through the expansion of some of the materials composing the slab, these materials contracting again when the water was allowed to dry off. He did not think the explanation was to be found in the compression of the imprisoned air. The last part of the paper was concerned with the micro-structure of concrete and cement, and particular reference was made to cavities of spherical shape. The author had said that he was not clear as to how the cavities were formed, but that from their spherical shape they were presumably caused by drops of excess water. Dr. Owens did not think that was correct. There was something else which occurred, and that was the imprisonment of air bubbles through the process of making the cement or concrete. Those air bubbles would take on spherical shape. He would not explain how, because they were all familiar with the effect of an air bubble in the liquid. The author had perfectly described those cavities, and it was certain, to his mind at least, that they were due to air bubbles. The variation in size was simply a matter of whether there was a large or small volume of air imprisoned. They might be found to be more numerous in the vicinity of particles of aggregate, and that might be explained by the carrying in of air with the aggregate. Dealing with the system of canals or ducts, Dr. Owens called attention to a peculiarity shown in one of the slides, and that was the tortuous shape of these. It struck him as remarkable that such a canal should be tortuous in one plane only. The sections were extremely fine, a thousandth of an inch perhaps. It was too much to ask that a tortuous canal should wriggle about in one plane,

and that they should happen to cut a section through that plane; he suggested that it was not a canal. There was another thing which he thought would occur, and that was the formation of small cracks. He had examined some of those cavities microscopically that morning, when he had observed that, and had found that there were certainly cracks. In fact, he was able to trace a crack entering a cavity of the kind under discussion, and by focussing on to the bottom of the cavity he could see the crack going across the floor of the cavity. There was another reason why he favoured the view that they were cracks, and not ducts, was that he could conceive of no possible process by which those canals would be formed in concrete; there were many reasons why cracks should be formed. The question of the effect of the viscosity of the water was one which should be taken into account in percolation experiments; they were dealing with very small passages and very slow rates of flow. The rate of flow of the liquid through the passages depended mainly upon the viscosity of the liquid, and the viscosity varied with the temperature, so that they had also to consider the temperature effect, or rather, change of viscosity due to change of temperature. He again thanked the author.

Captain Harrington Hudson said that when explaining the internal structure of concrete as revealed by the microscope, and in particular the spherical cavities, the author had stated that "these cavities are presumably caused by drops of excess water," and that "it would be worth while to see if the number of holes per square cm. is related to the percentage of water used in mixing." This was a point of considerable interest to the engineer, but how did the water content affect the absorption into and percolation through the concrete? An interesting series of investigation on tanks for oil storage had recently been carried out by Messrs. Pearson and Smith at the U.S. Bureau of Standards, Washington. Amongst the preliminary tests on water penetration losses were tests of three similar tanks. Each of these tanks was made of 1:2:4 concrete, but the mixtures were of different consistencies, i.e., 8 per cent., 9 per cent. and 10 per cent. water. He exhibited a lantern slide showing three curves plotted from data in the report of the tests. On this, losses due to absorption and percolation were plotted vertically, and time in days horizontally. The straight line running diagonally from the left top corner of the diagram

was the log plot of curve C up to 40 days, which proved that up to this time, curve C was truly exponential, but after 40 days the curve deviated from exponential.

The tanks were dry on the outer surfaces throughout tests of 140 days, so that the question of drip did not arise. He emphasised the fact that the water content in the mixture greatly influenced the absorption into and percolation through the concrete. Dealing with the author's reference to absorption into and percolation through a slab exposed on one surface only to water pressure, Capt. Hudson said the author had pointed out that the National Physical Laboratory tests showed that water actually passed through and dripped from the under side of the slab at a decreasing rate, although the cause of the decreasing rate was apparently temporary. The initial rate of drip was restored after partial drying of the slab. Later, the author had suggested that with more knowledge of the distribution of ducts in the concrete, this might be explained. Capt. Hudson said that he personally could not see how any distribution of the ducts could account for the phenomena. The temporary contraction of the ducts, due to swelling of the small grains under the action of water seemed to be a much more likely explanation. He then quoted the following from the report on tanks for oil storage, tested at the U.S. Bureau of Standards:—"In nearly every case the loss curve of any given oil in any given tank has become practically a straight line within two months after the beginning of the test." In other words, said Capt. Hudson, the rate of oil percolation ultimately became constant. The same could not be said in the case of water percolation, which gradually decreased. It appeared, therefore, that it was the action of water upon the concrete which accounted for the decreasing rate of drip, and that the arrangement of the ducts would not help to explain the phenomena. Another interesting development was recorded in the report. After about $2\frac{1}{2}$ months, fine cracks were discovered on the outside of the water tank, and these did not appear on any of the oil tanks. The tanks and their contents were kept at a fairly uniform temperature, and the only explanation which suggested itself, the report said, was that sufficient compressive stress (apparently due to swelling of the concrete) had developed in the wet concrete of the inside of the water tank to produce tension failure in the dry outer layer. The cracks could not be detected on the inside of the tanks. If concrete swelled gradually, under the action of water, said Capt. Hudson, the natural supposition was that the walls of the ducts or canals, being in direct contact with the water.

should swell first and thus tend to close up the ducts. He put this forward as a possible explanation of the decreasing rate of drip to which the author had referred.

Mr. C. T. Lewis said that with regard to the porousness of concrete, two interesting papers had been published, one by the Bureau of Standards, No. 58, which was most interesting from the water proportion point of view. When he had those papers about eighteen months ago he had a job in London where it was absolutely imperative that water should be kept out. He had made a series of experiments with the ballast and cement he was getting (the results practically agreed with the American tests), and had found that compression could be reduced by as much as half by an over-plus of water. He had worked this out to very fine proportions, and the work had gone on quite satisfactorily, which went to show that it was the water more than anything else in mixing concrete which was the trouble in causing porosity.

Mr. Ewart S. Andrews said the author should not have excused himself for presenting a paper which was principally scientific, because he believed they had all realised that matters which were perhaps only of scientific interest to-day might become of vital practical interest in a comparatively short time. In this particular paper, however, the practical interest was really very large. He was rather disappointed, however, that the paper was rather lacking in conclusions. It presented them with a number of interesting fields for experiment and enquiry, and he hoped the author would be able to extend the problems still further, in order that they might be able to derive more definite conclusions from them. He would like to see more distinction made beween penetration and absorption, and would also like to know the relationship between the absorption and the percentage of voids in the various mixtures.

Major Shingleton asked how the samples with which the author had experimented were mixed.

Mr. Moullin said they were very carefully mixed with a trowel, on glass.

The President said that Major Smith, who was present, had tried to get dense concrete by mechanical means. He had realised that density could be obtained by other means than a trowel and small specimens. Major Smith had endeavoured to get results with the efficiency of the laboratory and the mechanism of the mass production factory.

Major Smith said there was much to be said for trying to find a commercial means of getting laboratory results. They all knew it was possible to take various aggregates in small quantities, and so mix them that maximum efficiency could be attained, but commercially, the problem was not quite so easy. All sorts of things would slip in, and there was the human element to be contended with. There was undoubtedly a lot in the question of air bubbles in concrete; when turning over wet concrete, they undoubtedly turned in a quantity of air. As to the vibrating process, this not only vibrated the particles into position, but also shaped the smaller particles into the bigger cavities. By vibrating, they were inclined to push into those cavities smaller portions of aggregate, and afterwards, by applying pressure, they could get the particles into closer contact, closing the cavities and compressing the air there. The whole idea was to get to space as small as possible. There was still a lot to be done to get anywhere near the 100 per cent. efficiency mark, and he would like to see them all going ahead making experiments until they could get near that mark.

Mr. G. A. Gardner said that the paper was a valuable one, and it was hoped that the author would continue with his researches and deal with the important question of the percolation of water through concrete under stress. In all structures the concrete was under stress; and whether the canals were formed originally, or whether they were subsequent cracks, stress would have an effect upon them. He suggested, therefore, that perhaps the author could, in the experiment in which he used a tube pressed into the concrete, make the concrete, say, 18 inches span, and test the flow with the slab subjected to flexure under its own weight, and then with a small super-imposed load, and note the effect as regards percolation, because that seemed to be a very important point as applying to actual structures, for example, the bottoms of reinforced concrete tanks.

Mr. S. D. Carothers said he had made experiments on the percolation of air through concrete. He held that there must be a great number of cavities in the concrete to let out water, and that all those cavities must be inter-connected, and would take in further water.

Dr. Oscar Faber, O.B.E., wrote:—

I think this paper is a most interesting one, and I do not think the author need feel that it is more of a scientific than of a practising interest because the watertightness of concrete is one of its most valuable properties so far as reinforced work is concerned, since a concrete which is not watertight

is not only useless for reservoir work, but is dangerous for work exposed to atmospheric conditions owing to the risk of corrosion when it is not watertight.

During the war, I did some nine months experimental work for a branch of the Admiralty in connection with making concrete only 2 inches thick absolutely bone watertight under great heads of water, and eventually succeeded in making concrete only 2 inches thick alsolutely bone watertight under 100 ft. head of water. During these experiments the nature of concrete from the point of view of watertightness which the author describes with little air bubbles connected by capillary canals was fully investigated and confirmed.

The theory explained in the paper does to a large extent represent the mechanism, but undoubtedly it is only part of the mechanism, and there are various other important facts, without a knowledge of which one would get quite a wrong idea about the whole subject. It does not, for example, explain the fact that if a newly built reservoir of slightly porous concrete is filled and subjected to pressure from one side, the percolation is very rapid at first, and then gradually diminishes until in a few days' time it may have practically ceased.

According to the author's theory, once all the air has been expelled from the air bubbles and canals in the concrete a constant condition of steady flow should follow, but this is not found in practice to be the case, and in point of fact, the flow continues at a diminishing rate, and may eventually cease even when the water is practically free from impurities.

It is noticed in such cases when the percolation is reduced to a very small amount that a calcium salt begins to be deposited on the outside of the reservoir from which one is forced to the conclusion that on the inside face part of the cement appears to be dissolving, and no doubt the diminished flow is largely due to these salts blocking up some of the canals near the outside of the wall.

Another important phenomenon the author has, I think, omitted, and that is, that I think he makes a mistake in assuming the mass of the concrete to be unaffected by percolation, and to consider this as practically confined to the canals. The error of this, I think, follows from the undoubtedly experimental fact that when concrete is thoroughly wetted it expands quite considerably and subsequently shrinks again when dried. This, to my mind, clearly indi-

cates that the water has affected the body of the concrete, not only in the canals. It is also found that when the concrete is thoroughly soaked, the percolation is very much reduced, and this could be partly explained on the assumption that some of the dried colloid enclosed in between the grains of concrete expands considerably into a gelatinous mass and tends to block up the canals in very much the same way as particles of glue when wetted expand very much and would press against the sides of any containing vessel. I think it is important to bear in mind these other factors to get a proper idea of the mechanism of making concrete watertight, and of controlling percolation when it exists.

Major Leslie Shingleton, O.B.E., wrote: The first thing a practical user of concrete asks himself, after hearing this paper read, is: What can I learn from it to assist me in my work? At first sight, one is inclined to believe that it is not of much practical use, but on further consideration, this is found not to be so. True, laboratory experiments with specimens carefully mixed with a trowel on a sheet of glass, are rather difficult of comparison with the mixing in actual practice; and when one is constantly constructing water-tight tanks, with walls five or six inches thick, under a six to ten foot head of water, one cannot conceive that the laboratory specimens should allow water to percolate at all. It would be interesting to know the thickness of the slabs experimented with at the Physical Laboratory, and the composition of the concrete, but as these experiments were carried out ten years ago, they would not appear to be of great account at the present day in view of the increased knowledge and progress since that date. The one thing that appears to be emphasised from the reinforced concrete engineer's and contractor's point of view, assuming that Dr. Owen's opinion is correct; viz.: that the spheres seen under the microscope are caused by minute bubbles of air, and the so called capillary canals are fractures due either to the setting or in preparing the specimens—and I think his view is undoubtedly the correct one—is that in order to obtain impermeable concrete it is of the utmost importance to eliminate air, and in practice to pay particular attention to the placing of concrete in the forms, and working and punning same thoroughly; always presuming that the concrete is of the correct aggregate, right proportions, and thoroughly mixed; otherwise the presence of air voids probably connected by minute fractures due to setting or strains would undoubtedly cause permeability. This confirms the opinion

of most concrete users that properly proportioned, mixed and placed concrete should be watertight, and that the various solutions used for the purpose of making concrete watertight are supplementary, and only help to overcome minor defects in the form of minute fractures and voids, which in practice are present in concrete.

The PRESIDENT asked members if they themselves would carry out experiments with rgard to absorption. More information was wanted, and the text-books in the past had so often given, in the case of stones and bricks and similar materials, the weight of water absorbed after 24 hours. This weight of water had been expressed as a percentage of the original dry weight of the material. That might be useful for materials other than concrete, but in making watertight concrete it is necessary that there should be no unnecessary voids. For the purpose of eliminating voids they wanted to know what was the space which had to be filled with cement or other materials. He would give an instance of the disadvantage of the old orthodox way of expressing results.

Suppose they had two materials, one porous and light, and the other equally porous, but heavier. Let there be the same percentage of cavities in both, and let the cavities be connected with ducts or cracks. Suppose, in the first instance, in the case of, say, pumice stone, the absorption was 10 per cent. of the weight, and in the case of, say, slag or clinker, with the same amount of water absorbed if the material was twice as heavy, the absorption was only 5 per cent. That did not give them quite what they wanted, because the absorption was expressed in two different figures for the same volume of cavities.

He then called upon Mr. Moullin to reply to the discussion.

MR. MOULLIN, replying to the discussion, dealt with the difficulty of understanding why the water should drip from the under side of a slab at a decreasing rate, which had been referred to by both Dr. Owens and Captain Harrington Hudson. It was worth while, he said, to distinguish between absorption from the top and drip from underneath. He could not see how it could be explained in the manner in which they proposed it was possible. Absorption from the top was largely, he believed, a matter of dimensions of the blocks. With regard to the question of whether or not small spherical cavities were formed by air bubbles or by water, there was no proof of it being one or the other. It seemed to him that while a thing was in a fluid state, it was more likely that they would expect that if there were air bubbles

the air would easily escape. He had suggested that after it was set the resistance to the flow of air through a depth of a slab may affect the flow of water behind it. It was difficult for him to understand how that should be, say, in the fluid state. However, without further experiments it seemed impossible to tell one way or the other. Coming to the question of making allowance for variation of viscosity of the water with temperature changes the changes of temperature were not more than + or − 1 deg. The tests were carried out at ordinary room temperatures, and probably the range of temperature within the block was even less. It was probably rather a secondary matter to make an allowance for change in viscosity due to temperature. As to whether the ducts he had mentioned were actually ducts or cracks, he was not quite clear how Dr. Owens discriminated between the two. That is to say, if Dr. Owens had suggested that the crack had come about after the making of the microscopic slide—of course, that possibility had struck him long ago—he believed it was improbable that it was so, due to the position of the duct and the cavity in the slide, and also to the fact that they could see these ducts in the centre of a batch of set cement, not round a cavity, where one presumed they would come. However, if it were possible to prove that those things were cracks he would be very glad to know, because it was a matter upon which they could not decide one way or the other without definite proof. It was quite possible they might be actual ducts. Whether they formed during the setting of the cement, or otherwise, it seemed to matter very little. With regard to Capt. Harrington Hudson's curves, it was evidently established that the absorption and percolation curves were exponential to a high degree of accuracy. Such a point, he believed, had not been established before, and might possibly be of use. On the question of absorption of water into a block completely surrounded by water, the explanation offered by Capt. Harrington Hudson might meet the difficulty, but he would leave it to others to decide whether it was important or not.

With regard to the suggestion of Mr. Gardner that the concrete should be tested for percolation whilst under stress, it seemed to be a very valuable suggestion, and he would endeavour to carry out some tests of that kind.

A vote of thanks was accorded the author at the conclusion of the discussion.

THE ANNUAL DINNER.

The Annual Dinner of the Concrete Institute was held at the Savoy Hotel on Thursday, 2nd February, 1922, p.m., and was attended by 250 members and guests—a number far exceeding previous records.

A delightful evening was spent under the able chairmanship of the President, Mr. E. Fiander Etchells.

The speeches were of a high standard of excellence, and charming music was provided by the Misses Joyce Horton and Elsie Redfern, and Messrs. Lorne Wallett and Emil Clare.

Amongst those present were:—

The Rt. Hon. Sir Alfred Mond, Bart., P.C., M.P. (His M. Minister of Health).

The Rt. Hon. Lord Riddell and Lady Riddell.

The Very Rev. Dean Inge, C.V.O., D.D.

Sir Alfred and Lady Robbins.

Brigadier-General Sir Henry Maybury, K.C.M.G., C.B. (Director of Roads).

Brigadier-General Hon. F. C. Stanley, C.M.G., D.S.O.

Sir Henry Tanner, C.B., I.S.O. (Past President).

Sir Robert Robertson, K.B.E., F.R.S., and Lady Robertson.

Sir Henry Forman, M.P., and Lady Forman.

Dr. Cato Worsfold, M.P., and Mrs. Worsfold.

Mr. H. D. Searles-Wood, V.P.R.I.B.A. (Past President) and Miss Searles-Wood.

Mr. Topham Forrest, F.R.I.B.A., Hon. M.C.I. (Superintending Architect of Metropolitan Buildings), and Mrs. Topham Forrest.

Captain H. Riall Sankey, C.B., C.B.E. (President of the Institution of Mechanical Engineers), and Mrs. Riall Sankey.

Mr. E. J. Sadgrove, F.R.I.B.A. (President of the Society of Architects), and Mrs. Sadgrove.

Mr. J. Forsdike, F.I.O.B. (Past President of the National Federation of Building Trades' Employers).

Mr. Roland B. Chessum, F.I.O.B. (President of the London Master Builders' Association), and Mrs. Chessum.

Mr. Ewart S. Andrews (Member of Council), and Mrs. Andrews.

Major A. Boyd-Carpenter, M.P.

Mr. W. E. A. Brown (Member of Council).

Mr. H. F. Bladen (Member of Council).

Mr. D. B. Butler (Member of Council).

Mr. H. J. Deane (Vice-President) and Mrs. Deane.

Mr. H. Kempton Dyson (Member of Council), and Mrs. Dyson.

Mr. W. A. Green (Member of Council).

Engineer-Commander P. F. Griffiths, C.M.G., R.N.

Mr. J. E. Franck (Vice-President) and Mrs. Franck.
Dr. Oscar Faber, O.B.E. (Member of Council) and Mrs. Oscar Faber.
Mr. G. C. Humphreys, C.B.E. (Chief Engineer of the London County Council).
Mr. E. L. Hall (Member of Council).
Mr. B. L. Hurst (Member of Council).
Major Alfred Harris, D.S.O.
Capt. M. G. Kiddy (Secretary) and Mrs. Kiddy.
Mr. W. J. H. Leverton (Member of Council) and Miss Leverton.
Mr. C. F. Marsh (Vice-President).
Professor E. R. Matthews (Member of Council).
Mrs. M. G. Morris.
Dr. J. S. Owens (Vice-President).
Major James Petrie, O.B.E. (Member of Council).
Lieut.-Col. G. W. Parkinson, M.C.
Mr. F. Purton (Member of Council).
Lieut-Col. F. C. Rogers, C.M.G., D.S.O.
Engineer-Commander Fraser Shaw, R.N.
Mr. H. E. Steinberg (Member of Council).
Mr. F. E. Wentworth-Sheilds, O.B.E. (Past President).
Mr. G. C. Workman (Member of Council).

The President, in introducing the first toast, epitomised something of the tangled history, and the present solidarity of the British race, in a single sentence, for his first toast was "To the King, the King of Great and Greater Britain; the Sovereign of the British Commonwealth of free nations; the overlord of Pict and Scot, and Fein and Sassenach."

In proposing the second toast, the President referred to the Prince of Wales as the "bright reinforcement of our concrete Empire," and expressed the hope that H.R.H. Princess Mary would after her forthcoming marriage be the proud possessor of a concrete home of her own.

Before proposing the toast of "His Majesty's Ministers," the President announced the receipt of letters of regret for their inability to attend from H.R.H. the Duke of York and from H.R.H. Prince Henry, from Mr. Asquith, Sir Aston Webb, President of the Royal Academy, and from the President of the Royal Institute of British Architects (Mr. Paul Waterhouse), and the President of the Institution of Civil Engineers (Mr. Barton Worthington), who had gone north on important business.

Sir Charles Ruthen, O.B.E. (Member of Council), who was to have proposed one of the toasts, was prevented by illness from attending.

In giving the toast of H.M. Ministers, the President reminded the company that in having the Minister of Health with them they had one of the hardest worked Cabinet Ministers. If other members of the Cabinet had work with which they could not deal, they handed it over to the Minister of Health, whose duties covered such widely-differing subjects as nursing and reinforced concrete regulations; it would be his duty to sign and approve the reinforced concrete regulations—whenever they should be revised—and therefore he would no doubt have to study the inner meaning of such things as the ellipse of inertia, the modulus of a section and gyration radii about several inclined axes. In continuing the President stated that he had had a terrible dream. He dreamt that Sir Alfred Mond was dead, and that he had gone to a heaven—the special heaven reserved for Cabinet Ministers, and as Sir Alfred Mond went along the golden pavements he saw a great mass of white crystals. He knew that the material was not snow. His profound knowledge of industrial chemistry enabled him to recognise it as being sodium chloride. He asked St. Peter why it was there, and was told: "We are expecting some *more Monds* shortly, and we want to make the place look like Salt Lake City!" (Laughter.)

But amidst their laughter, they must not forget the seriousness of the duties and the responsibilities of the Cabinet, for upon the Cabinet depended the destiny of the Empire. They therefore trusted that the decisions of the Cabinet would be wise and would consolidate the position of the Empire, which had passed and was passing, and must still pass, through times of trouble and of stress. It was their sincere and fervent prayer that the consultations of the High Court of Parliament and the Cabinet might be directed to the "safety, honour and welfare of our Sovereign and his Dominions; and that all things might be so ordered and settled by their endeavour upon the best and surest foundations, so that peace and happiness, truth and justice may be established among us for all generations."

This is our prayer, but what of our duty. Is it not that amid the clash of divergent interests within the State, we should yet strive for the good of all, so that we may work and live in a land

> Where freedom's "form may stand and shine,
> Make bright our days and light our dreams,
> Turning to scorn with lips divine,
> The falsehood of extremes."

At the close of the President's remarks the toast to the Ministers of His Majesty's Coalition Government was received with enthusiasm.

In responding, Sir Alfred Mond, Bart., P.C., M.P. (Minister of Health) said he had to thank the President for the very kind and felicitous terms in which he had proposed the health of the Ministry of which he had the honour to be a member, and especially to his kind personal references to himself. He also thanked the gathering for the very enthusiastic manner in which they had responded to the toast. He was glad to think there was at least one man in this country who was so impressed with the burdens of a Cabinet Minister that he allowed it to disturb his sleep, but he could not say how relieved he was after the President had begun his story to find that he had ascended to a higher plane, and that the President had not invented a specially-reinforced lower place.

It was very satisfactory to know that a member of a Government which was much maligned, much criticised, but seldom praised, should be treated with such distinction as he had been treated by the President.

He thought the office he had the honour of holding in 1916, as First Commissioner of Works, brought him perhaps into closer touch with the subject in which they were all interested than the office he now held. He did not want to launch out into a panegyric of concrete, or reinforced concrete and modern construction, for he knew a good deal about its importance, and that in the Office of Works they had a trained expert to deal with that branch of the work —and when a Government Department had reached the pitch of employing an expert in any industry, that industry "had arrived!" He thought in the period after he took office the Office of Works erected a greater number of large concrete buildings, and even coke-breeze partitions, than almost any other department or firm in the history of the world. They were engaged in those days in the erection of the enormous warehouses, aeroplane sheds, shell-filling factories, and all those adjuncts to the carrying on of the great war which filled their days and nights with anxiety, and the end of which had been such a relief to a suffering humanity.

He was now occupied more with the cares of peace—such peace, at any rate, as they were permitted to have—and they all wished there were more peace throughout the world than there was to-day. It must be the ardent desire

of everyone, apart from party, apart from nation, apart from all other interests, to see again a calm sea and blue skies, and the ship of State pursuing her course on an even keel across untroubled waters. Such, alas, was not the position yet, either in Europe or in other parts of the world, and it was but little realised what a heavy swell remained on the ocean of the world from the last enormous gale, what lowering skies there were from which the sun only fitfully appeared at times, and how that impeded and made difficult the task of those who had some responsibility, either as captain or crew, to steer the ship of State at the present time.

He had said that the Government was much maligned and little praised; that, he thought, was the case with all Governments throughout history. It was the duty of critics to criticise, but he often wished critics would obtain a little information before they instructed the world what to do. If those people would go to despised Government departments to obtain some elementary facts they would do their country better service. But the whistling of that kind of storm went through the rigging, and did not hamper the progress of the ship of State very much. The ship of State was a good old ship—a ship built in the days when the people built with such stout timbers that the ship had weathered the greatest gale in its history in a most magnificent way.

We were too modest as a nation, and people were, he thought, unduly depressed. We had to go out of our own country in order to realise the position this country really held. When one got abroad among other nations, how often one appreciated much more the steadfast, courageous, and calm manner in which this country had just passed from victory to calm, from war to peace. Whatever people might say, however despondent people might be, because we had not yet got back to normal conditions they must cast their minds back to the time of the Armistice and realise the difficulties that had been overcome. Vast armies had been disbanded, and had returned to civil life; huge industries created during the war had been disbanded and replaced with peace industries; the period of huge borrowings had been passed, and a normal budget that was paying its way had been reached—and this was the only country in Europe that was doing so. Those who were maimed in the war had been looked after in a manner without parallel in any other country.

If one looked back calmly and quietly, which he had no doubt the members of the Institute were in the habit of doing, he thought the Government could claim that it had done a stupendous task, and there had been one Government which had been common throughout all those difficult times. He did not claim that the Government of which he was a humble member, faced with circumstances unparalleled, with an uncertainty of the future, had not made mistakes, or had not had to retrace its steps or to shape its course according to circumstances. But he did claim that on the whole it had brought this country through its difficulties with success, and that the credit of Great Britain stood higher in the world to-day than at almost any period of its history

Stability had its advantages, and those of them who erected magnificent reinforced concrete buildings knew that if they collapsed it did not increase the designer's reputation, or help on the cause in which they were interested. He saw in the paper that evening that the new Italian Government had resigned. A short time ago the French Government resigned. Since the Peace of Versailles it would be quite a formidable task to count the number of Governments and Premiers who had resigned. It was only in this country that one Prime Minister and one Government had continued to conduct the affairs of the country since the Armistice, and that was largely due to the common sense of the British people, who had realised, and he thought rightly realised, the enormous advantage of allowing those who knew the difficulties from beginning to end to continue to represent them in the councils of the world.

One had only to look to Washington, where a conference unequalled in importance in the history of humanity had been held, with results so wide-reaching for the future that they were only dimly apprehended. That conference, called by the initiative of an enlightened American Government, had immensely strengthened and supported the policy of the British Government, the work of whose distinguished representative, Mr. Balfour, had been so much appreciated. The task was difficult, and the help and assistance of those who wished well of their country was required. He did not believe there was one member of the Government who would not be glad and ready to leave the burden of responsibility to other hands. Honourable and dignified as was the position of a Minister, it would be idle to discuss the fact that it carried with it incessant toil, heavy responsibilities, and

continuous insight as to what the future would hold. He was an optimist during the war, and never had any doubt as to its result from the day it broke out. We had passed through the war and come out victorious, and he believed we could pass through as difficult times again and come out even better than we had in the past. Two things were required, one of which was patience. We lived in a time when everything moved very quickly: we motor-scooted along the roads, aeroplaned through the skies, films passed before our eyes, and we had scarcely a meal without it was at a quick-lunch counter. We bought three papers every day, and expected to find something different in each one of them—and sometimes the editors provided it for us! We were apt to be impatient, but the disturbance had been so great that time must be allowed for recovery. People had been too distraught for a calmer view to be expected at a moment's notice. But he thought the world was improving, and the general recognition of difficulties was becoming more and more apparent. He had hopes that a conference of a most important character would discuss many considered interests and weld them into one common interest. This country stood as a firm rock in the middle of a quaking world.

England was a building the Concrete Institute could well be proud of. It stood firm on its foundations, reinforced right through, with no weak trusses, no breaking stresses, no tendency to sag, no cracks developing in frosty weather, and no water to penetrate in a storm. It was tight, comfortable, cosy and strong. And the world looked to England to keep it so. And the Government, so far as it could, would bend all its energies to maintain it, and with the assistance and the help and sympathy of those for whom it laboured and felt itself responsible to, he had no doubt it would succeed.

Lord Riddell proposed the toast of "The Concrete Institute," and in the course of his remarks said perhaps they would wonder why he was there, and he was bound to confess that he rather wondered himself. The explanation, however, was very simple. While he was in America he received a cablegram informing him that he had been appointed President of the Gas Producers' Association, and being a newspaper man he need hardly say he regarded that as a somewhat doubtful compliment; indeed, in the first instance he rather thought it might be a malicious joke. However, the use of the cable enabled him to ascertain that he had been appointed to the post on his merits. He under-

stood he had been invited there that night because he held the distinguished position of President of the Gas Producers' Association, although he did not know what gas production had to do with concrete; he supposed, however, there must be something not apparent to such an unsophisticated person as himself. He knew enough about concrete to know that it was a very tough subject, but until he read Mr. Etchell's Presidential Address he did not realise its extraordinary ramifications. The Institute could boast a most able Secretary. Their Secretary had wonderful organising ability. They had got a very admirable Secretary, and when he saw such men, who had been away at the war, and had come back to fill positions such as their Secretary was filling with such distinction, he felt proud of them. The previous night he had received a copy of the "Journal of the Concrete Institute," containing the Presidential Address, which he read very carefully. After reading it he saw at once what a complicated and diverse subject concrete was, because he found that the President referred, amongst other topics, to education, capital and labour, inspection and supervision, and the fire-resistance of wood (he said some nasty things about wood!) The President then got on to bricks, and was very severe in his remarks. He thought the President dealt with every other material used in building construction, and had nothing good to say about any of them except concrete.

The President then dealt with relativity, but that was a very dangerous subject to tackle, because relativity implied bending, which concrete did not. He had carefully perused all the literature sent him, and believed at that moment he knew more about the history of the Concrete Institute than any other living man. They might be surprised to learn, having regard to the youthful appearance of the members present, that the Institute was founded as far back as 1908, and, needless to say, it was what might be called a "concrete centre." He had had a good deal of experience of the English Technical Institutions, and he had for long been of the opinion that such bodies formed one of the most useful and admirable phases of the professional and industrial life of the country. He had been in close contact with many of them, but he knew of few institutes or few associations which possessed such a capable and energetic Secretary as did the Concrete Institute. Few of them had Secretaries who sent so much literature out, in fact he had a library of Concrete Institute literature, which

he would read when he felt dull! He had discovered the surprising fact that one could not touch any phase of human activity, whether it be gas, or concrete, or engineering, or electricity, without finding that our countrymen always stood at the top, and they always had stood at the top. When they made him President of the Gas Producers' Association he was in America, and when he got tired at the Washington Peace Conference he turned to the papers the Association sent him and said, "Here is a little light reading." And from those papers he found that for the past 100 years there had been brilliant, able Englishmen—of whom most people had not heard—making surprising inventions and doing wonderful things without any self-advertisement, and, he expected, for very little money, in the industry with which they happened to be concerned. He found that the leading books on the subject of concrete were written by Englishmen or Irishmen—he also thought Scotchmen would be anywhere where there was concrete. Our country had always been in the forefront in the matter of concrete, as in other industries. Unless one had been away, one did not properly appreciate what wonderful people English men and women were. The President of the Concrete Institute was a typical British professional man, and he could not pay him any higher compliment than that—it was no good trying to gild the lily! The issue he had received of the *Journal* of the Institute gave an account of the President's career, and he saw that he was elected a member of the Institute in 1908, and in 1920 he was elected President, and he held no less than twelve different offices, all of which he seemed to have held with great distinction. He also belonged to the Royal Institute of British Architects, the Institution of Civil Engineers, the Institution of Mechanical Engineers, the Institute of Physics, and the Mathematical Association—in fact there was hardly any scientific society to which he did not belong. He was a worthy and energetic President, and not only a man of action, but had great powers of exposition.

In responding, The President thanked Lord Riddell in the name of the Institute, and concluded with an entertaining account of the manner in which concrete was made by the Trades Unionists in Utopia.

Mr. J. Ernest Franck, F.R.I.B.A. (Vice-President), in proposing the toast of "The Guests," said the members wished to give the best welcome they could and the heartiest greeting to their visitors. The Concrete Institute, which had been told that evening that it represented professionally

the best race on the earth, hoped its guests had received a fitting welcome in every respect; it wished them every good wish. He was coupling with the toast the names of Dean Inge and Sir Alfred Robbins. Dean Inge represented the Church, and he also was representative of reinforced concrete. Concrete, like the Church, had existed for thousands of years. Dean Inge represented the modern application of the Church to the conditions of to-day in exactly the same way as reinforced concrete was the application of materials in use thousands of years ago, but which were used in combination to-day. Sir Alfred Robbins was a public man who used his influence for the best things in this country, and he was a man they were proud to see there that night.

The Very Rev. W. R. Inge, C.V.O., D.D. (Dean of St. Paul's), in reply, said he could not claim to know very much about concrete. As a student of philosophy his own interests had been in the "abstract," rather than in the "concrete." He did not know very much about the uses of concrete. Two certainly, were the emplacements for heavy guns, and the foundations for winter lawn tennis courts. He hoped that in the future the activities of the Institute would be devoted more to the second of these than to the first. In conclusion, he thanked the Institute, on behalf of the guests, for the exceedingly pleasant and delightful evening the members had given them.

Sir Alfred Robbins, who also responded to the toast, said he had been asking himself, "Who am I to speak after the Dean?" For the Dean held a position unequalled for influence on the lives and thoughts of the citizens whereas he himself could only claim to be a humble member of that great society of gas producers of whom Lord Riddell had become the President. As a diplomatic agent and an agent of publicity Lord Riddell stood as one who had done most to secure to the public of this country the communication of things of importance regarding public events, and he had done most to let us know what happened at those great international conferences which were bringing good fruits. Turning to the objects of the Institute, Sir Alfred remarked that in passing along the streets of London one sometimes saw suddenly emerging from the abyss great skeletons of steel, and one wondered to what Government department they would appertain. Then there came along the concrete which rendered them beautiful and strong and fit to live in. He admired the way in which such buildings rose, and avoided the thought that in some other countries they sometimes fell. He thanked the Concrete Institute, and wished it still higher success in the future.

PROCEEDINGS.

The 108th ordinary general meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, January 26, 1922, at 7.30 p.m.

The chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous meeting, as published in the *Journal*, were taken as read, and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association:—

Members.

BASTO, ANTONIO H., 46, Cambridge Road, Battersea; HUGHES, JAMES CLEMENT, Industrial Constructions, Ld., 44, Grosvenor Place, S.W.; MOBBS, STUART KILSBY, Messrs. Holloway Bros., Ld., 157, Grosvenor Road, S.W.1; THOMERSON, ALBERT, 9, Finsbury Square, E.C.2; HIGGINS, FRANCIS COLEMAN, 9, Finsbury Square, E.C.2; *HAMLIN, ERNEST JOHN, P.O. Box 17, Stellenbosch, South Africa.

Associate Members.

BAIGENT, JOHN WALTER, 114, Ivy Road, Cricklewood, N.W.2; CROSBIE, JOHN, 76, Chadwick Street, Windsor; FIRTH, HARRY, 15, Etruria Street, Manchester; GRIFFITHS, SAMUEL PRICE, Hadley, Alderley Edge, Cheshire; HOLDEN, EDWARD, Technical Schools, Gloucester; PARR-HEAD, ARTHUR SYDNEY, Derwent Yard, Ealing, W.5; ROBIN, ROWLAND CUTHBERT, 92, Oatwood Court, Kensington; SENIOR, EDWARD POWELL, Via y Obras, F.C.O. Estacion Once, Buenos Aires.

The secretary announced the reinstatement to membership by the Council of J. SCOTT MARSHALL, of 11, Broadwater Down, Tunbridge Wells.

Resignations.

The Secretary announced the reinstatement to member- of the resignations of PERCIVAL STEVENS, member, elected in 1909; and of F. E. L. HARRIS, member, elected in 1909.

Deaths.

Great regret was expressed at the report of the deaths of Colonel W. D. REES, Member, elected in 1911; and of FRANCIS DRING, Member, elected in 1912.

A paper was then read by Mr. E. B. MOULLIN, M.A., on "Capillary Canals in Concrete and the Percolation of Water through Them."

A lengthy discussion followed (reported elsewhere), and a vote of thanks to the lecturer, proposed by DR. OWENS, and seconded by CAPTAIN HUDSON, was carried with acclamation.

MR. MOULLIN having replied, the proceedings terminated.

* *Transferred from Associate-Membership.*

THE CONCRETE INSTITUTE
PUBLICATIONS.

"Transactions and Notes."—

Vol. IV., Pts. 3 and 4	(Each)	2s. 9d.
,, V., Pt. 1		5s. 6d.
Pt. 2		2s. 9d.
Pts. 3 to 6 (inclusive)		11s. 0d.
Vols. VI. (Pts. 1 and 2), VII. and VIII.		5s. 3d.
,, IX.		5s. 9d.

"Standard Notation for Engineering Formulæ" ... ... 6s. 3d.

In this Report the Science Committee provide a system of standard symbols in order that time and consequently money may be saved in writing down the calculations which all classes of engineers are making daily in following their professions. The idea is to provide means whereby mathematical equations can be typed more easily, and printed more cheaply, and also to lighten the students' task by providing a set of self-explanatory symbols which will become imprinted upon the memory almost automatically, and be retained there without effort.

"Report of Joint Committee on Loads on Highway Bridges" 2s. 8d.

Effect of speed increasing virtual load; Table of standard loading for bridges of three classes; diagram of typical loads; tractive efforts on various road surfaces; notes on heaviest loads occurring in different districts; bending moment diagrams for standard loading; shearing forces; calculated example; effect of width of tyre and diameter of wheel; maximum axle loads allowed in various districts: Acts and Orders relating to motor traffic; scale diagrams of heavy vehicles with Table of loads and dimensions; determination of bending moments and shearing forces for beams subjected to moving loads.

"Recommendations to Inspectors, Clerks-of-Works and Foremen concerning the execution of Reinforced Concrete Works" ... 6½d.

"Standard method of measurement of Reinforced Concrete in Building construction," as adopted by the Concrete Institute and the Quantity Surveyors' Association ... 1s. 1d.

This pamphlet contains a full description of the order in which the work should be taken, the items which should be measured and grouped separately, and the mode of measurement in each case for *in situ* and *pre-cast* work.

"A Standard Specification for Reinforced Concrete Work" 1s. 7d.

Reports to the Scientific and Industrial Research Department on the Research Work on Concrete carried out under the direction of the Concrete Institute (1917-1919) ... ... 1s. 7d.

Post Free (Inland).

MEETING IN APRIL.

The Annual General Meeting of the Concrete Institute will be held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, April 27, 1922, at 7.30 p.m., when the Council's Annual Report for the year 1921-22 will be presented.

Following this meeting will be held the 111th ordinary general meeting, at which a paper (illustrated with lantern slides) will be read by Mr. E. F. Sargeant, A.M.Inst.C.E., M.I.Mech.E., on "The Preparation of Concrete Aggregates" and "Moving Forms."

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS pay	First 500 ...	£1	1s.	0d.
	Remainder ...	£2	2s.	0d.
ASSOCIATES	... ...	£2	2s.	0d.
ASSOCIATE-MEMBERS	... ...	£1	11s.	6d.
GRADUATES, STUDENTS ,, ...	... ...		10s.	6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

CONCRETE INSTITUTE MASONIC LODGE.

Some members of the Institute decided, at a meeting held on March 9, 1922, to present a petition to Grand Lodge for the founding of a Concrete Institute Lodge. Members interested should communicate with Captain M. G. Kiddy, at 296, Vauxhall Bridge Road, S.W.1.

EXAMINATION IN MAY.

Examinations, Part I. for Graduateship and Part II. for Associate-Membership, will be held in London and at other centres as may be arranged, on Thursday and Friday, May 18 and 19.

A Syllabus and full particulars may be obtained on application to the Secretary of the Concrete Institute, 296, Vauxhall Bridge Road, S.W.1.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. Incorporated 1909.

Vol. XII. *APRIL, 1922.* No. 4.

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

THE ANNUAL GENERAL MEETING.

The Thirteenth Annual General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W. 1, on Thursday, 27th April, 1922, at 7.30 p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous Meeting were taken as read and were confirmed.

The Annual Report and Balance Sheet of the Institute were presented by the Council, and on the proposition of MR. W. J. H. LEVERTON, were unanimously adopted.

On the proposition of MAJOR PETRIE, Chairman of the Finance Committee, the Auditors, Messrs. Monkhouse, Stoneham & Co., were re-elected for the ensuing year, on the usual terms.

The announcement of the re-election of Mr. E. Fiander Etchells as President for a further year of office (commencing with November) and of the Council elections (reported elsewhere) was received with acclamation.

A detailed report will be published in due course.

WHAT IS THE USE OF THE MODULAR RATIO?

By H. KEMPTON DYSON.

The title of this paper does not properly describe its scope. It savours perhaps of the frivolous and the ostentatious, but it was chosen because it indicated shortly its purpose of throwing doubt upon the current and generally accepted theory of reinforced concrete calculations in which the so-called "modular ratio" is so fundamental a factor Of course, the ratio of the moduli of elasticity of different materials enters into calculations as to the relations of such materials when subjected to static or dynamic forces.

The modulus of elasticity expresses the relation between stress and strain, and is the ratio thereof. Expressed in simple mathematics:

$$f = \frac{F}{A}$$

$$\sigma = \frac{\triangle}{l}$$

$$\frac{f}{\sigma} = E$$

where A = area
$\triangle$ = Deformation (in units of length)
E = Elastic modulus
F = Force
l = length
f = stress (in units of force per unit of area)
σ = strain (in units of lengths per unit of length)

The strain may also be measured volumetrically by angular distortion, but that is not here considered for the sake of keeping to simple explanations.

The modulus of elasticity is often referred to as though it were a constant, i.e., the ratio of the stress to the strain as uniform. For almost all materials the ratio is variable.

Steel is a material which, for low stresses, seems to have almost a constant modulus, the point where there is a distinct departure being known as the elastic limit. At a slightly higher stress there is a very distinct departure, the strain increasing greatly and being largely plastic, giving a "permanent set." This is known as the "yield point."

In many practical constructions a certain amount of per-

manent set is always reckoned upon. With rivetted joints, for example, some of the rivets have to give way slightly before the others are brought into play. In comparatively roughly cut wooden joists resting upon wall plates which in turn rest upon rough brickwork, there must be some permanent deformation before they all come into proper bearing. Even in brick and stone work the joints are laid up in soft mortar in order that there may be plastic deformation. Considerable movements are often allowed in brickwork. In ordinary building, therefore, we are familiar with permanent set, and we feel quite confident as to the stability of the construction when it settles down to its work.

The computation of the stresses in materials subjected to permanent plastic deformation has not received sufficient attention. Elasticians (as those scientists who have made notable contributions to the mathematical theory of stresses and strains in elastic materials have been called) have simplified their mathematics by considering only ideally elastic materials—which would imply ideal conditions.

Fig. 1 shows a stress-strain diagram of a tensile test on ordinary mild steel stretched to beyond the yield point. The curve shows that when the stress is taken off the bar and the load falls to zero, there is a permanent set. The loading is now repeated. For this second time of loading the modulus (ratio of stress to strain) remains practically constant up to the limit to which the bar had first been strained. If the loading be continued beyond that point, there is again further plastic deformation, and, on the removal of the load, a similar result occurs.

This permanent stretching can be shown in another way. Fig. 2 is a stress-strain diagram that has been produced by Mr. J. A. Wicksteed. In obtaining this curve the loads were applied by means of a testing machine in which the pull fell off as the bar extended if it were let rest (a hydraulically operated machine, for instance). A bar can thus, by its elongation, reduce the stress to which it is subjected, and when the yield point is exceeded it will be found at points, if one periodically refrains from continuously increasing the load, that the bar will extend in the periods of rest so that a balance is established between the load and the strength of the bar. It is asserted by Mr. Wicksteed that the load obtained in this way can be withstood by the bar permanently for any length of time; for instance, in the curve shown the load was applied up to the amount represented by " B," the curve being approximately straight up to the point " A," which is usually termed the elastic limit.

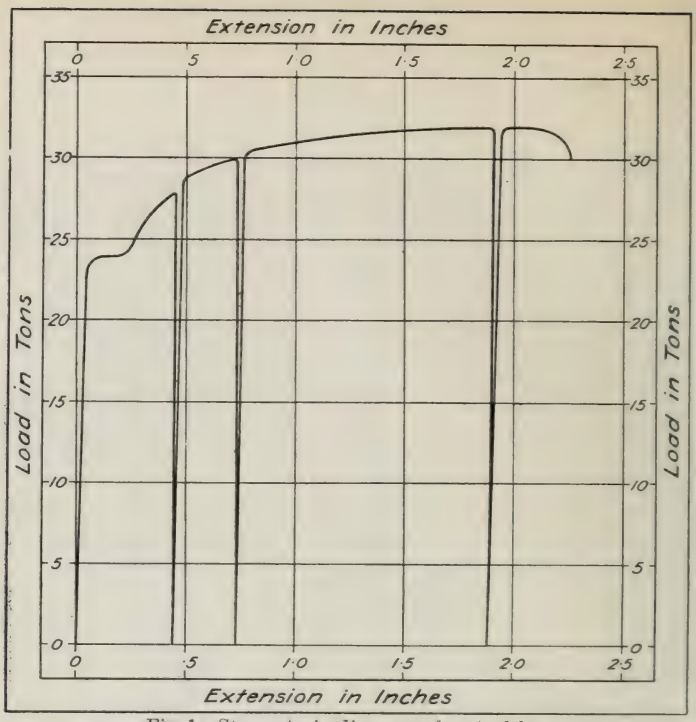
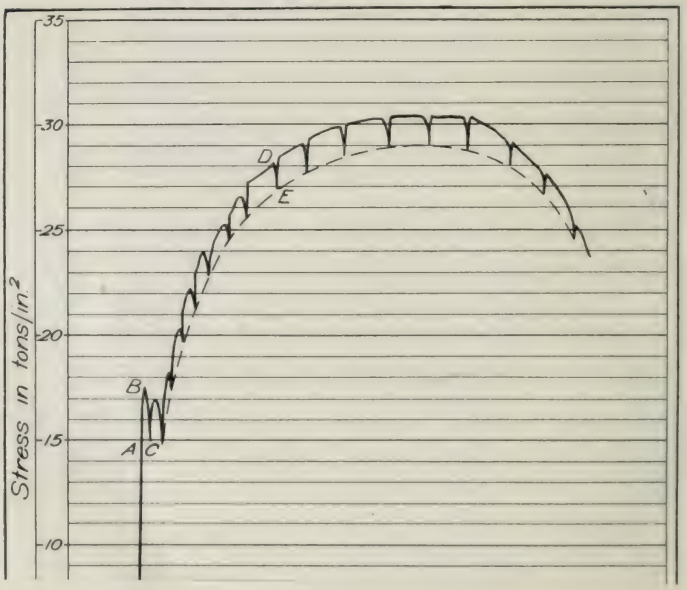


Fig. 1. Stress-strain diagram of a steel bar.



At the point " B," the material yields and the curve drops to the point " C," at which the stress can be withstood by the bar without further deformation. The point " C " represents the normal elastic limit. Now if the straining be continued, as the bar stretches it will get stronger and not more plastic; that is to say, the material will require a greater stress to make it flow, and the bar will harden with the continual yielding. If a stop is made at a point such as " D," the yielding will cause the stress to drop to " E," when it will again stop, and that stress will be the statical stress which the bar will withstand under such conditions indefinitely without further extension. The point " E " is therefore a new elastic limit. At the top of the upper curve shown in the diagram, while the steel has an apparent strength of 30.5 tons, the statical strength is actually 29 tons. In drawing a curve of this kind it is to be remembered that the stress shown is to be measured upon the ordinary section of the bar, and that on straining the section becomes reduced, so that although the apparent stress drops down as shown by the downward sweep of the curve towards the end, yet the real stress on the reduced section would be continuously increasing until possibly 50 tons would be the actual stress at failure. This is the explanation of the high ultimate strength that can be obtained from cold-drawn steel wire if it has been much worked, although the original material may have been soft with a low yield point.

It will be seen that with ordinary ductile mild steel, if we exceed the normal yield point, we may create new and higher yield points by cold working the steel, and so long as any repeated loading does not exceed the new yield point, no further permanent set is to be feared.

Concrete has no well-defined yield point, like steel; but somewhat similar phenomena are found by causing the material to undergo permanent deformation by repeated loading.

A number of diagrams of this kind were given by Professor Lea in a paper on the " Modulus of Elasticity of Concrete," which he read before the Institute on April 28, 1921, to which reference should be made. By repeated loading the modulus of concrete becomes much more uniform than appears from the stress-strain curve for the first time of loading, but, nevertheless, the modulus is not constant. Professor Lea, in his paper, pointed out that there were two ways of referring to the modulus of elasticity of a curved stress-strain diagram, namely:

(1) The average for a certain limit of stress. For instance, if tested in compression up to 1,000 lbs. per square

inch, that stress is divided by the total strain up to that point in order to obtain the value of the modulus of elasticity.

(2) The instantaneous value at any particular stress. This is derived by taking the ratio at which the stress and the strain are increasing at any point, and is shown by the tangent to the curve at the point.

To distinguish between these two ways of determining the modulus of elasticity we may point out that concrete which may start, for example, with an initial modulus of elasticity of about 4,000,000 pounds per square inch will soon drop off successively to three million, two million, one million and 500,000 pds./in.^{2}. If we take a point, say, where the instantaneous modulus determined by the tangent to the curve is, say, 1,000,000 pounds per square inch, if the stress be divided by the total deformation from the start to that stress a value of 2,000,000 pds. per square inch may be given. This shows the difference between the instantaneous modulus and the average modulus. Generally, for ease and comfort, the modulus is treated as being constant, and for concrete the " average modulus " is adopted.

The author has already, in a paper on the subject of " Shear," read before this Institute, made some observations on the distinction between brittle and ductile materials. He mentioned therein that certain engineering experimentalists and theorists were of the view that the failure occurred in ductile materials when the tangential or shear stress in the material reached a certain value. It was there pointed out that steel is composed largely of crystallised substances, and when deformed beyond the elastic limit these crystals break up into smaller ones, sliding taking place on the cleavage planes of the said crystals. Some elasticians hold the view that failure in such ductile materials occurs when the strain reaches a limiting value, but Coulomb—followed by Mr. J. J. Guest—holds the view that failure occurs when the shearing stress reaches a limiting value. It is rather peculiar that slipping can take place in crystals, and yet the faces of such crystals remain strongly attracted to each other. It would seem from the fact that when steel is deformed and the crystals become smaller, that the attraction must be inversely proportional to the size of the crystals.

Another distinctive fact, in regard to the crystalline state is that the particles of matter appear to be arranged in a definite order, so as to give directional properties to the material, i.e., in one direction its properties will be different from another; thus, for instance, it may be harder or it may conduct light, heat, or electricity better, or may turn a ray of

light at a greater angle. Such materials can generally be beaten or squeezed out of shape without losing their tenacity, and they appear to act like solid fluids. As illustrating this point, reference may be made to how a compression cylinder of steel will continuously deform without fracture; it will be found, when it is thus squeezing down into a sort of barrel-shape, to have a constant intensity of pressure per square inch of the cross-section. This has been called the " pressure of fluidity," and, in mild steel, is somewhere in the neighbourhood of 50 tons per square inch.

In the author's former paper on " Shear," attention was drawn to the fact that round bars of such ductile materials, when drawn apart in tension, developed a stricture, drawing down to a sort of a waist and showing a cup-like fracture, whereas materials of different category did not. Opposed to ductile materials are brittle ones. There seems to be a class of substances, however, to which no previous reference was made, that is, those that may be ductile or brittle yet appear to have no directional properties, as, for instance, indiarubber, jelly, glass and the like. Indeed, these substances seem to conform more in certain of their properties to the ideal substances imagined by the elasticians in that they have no directional properties. One cannot say, however, that they are ideally elastic—indeed, such materials often have a considerable degree of plasticity.

The author, in his former paper, in considering structural materials that were brittle in cross-bending pointed out that stone, mortar, concrete and cast-iron were composed of hard particles embedded in cement. Detailed consideration was given therein to the nature of such materials and formulæ put forward, treating them as granular substances with cohesion between the granules, such cohesion, of course, being due to the cementitious material in which they were embedded. In such substances the granules must be harder than the cement. Clay also is of the same kind. Fracture occurs by the cohesion being overcome. The failure thus takes place by tension.

There are, of course, other brittle substances which are not granular in the same sense. They may be without directional properties, like glass, but some few are not. Cases in point are hard, fine-drawn steel wire and tool-steel. They are elastic though not ductile, but their kind of brittleness is of a different order.

Ductile materials can be plastically deformed to a large extent without any very great deterioration as regards their strength. Indeed, ductile materials are frequently increased in strength by such plastic deformation. It is found, how-

ever, that by heat treatment the small crystals that were formed by the cleaving of the original big crystals can grow back again to the original size, or, in other words, the small crystals grouped together again flow into each other, and recrystallise as larger ones. That is what happens in the process of annealing We are all familiar with the fact that bright drawn steel wire has a greater tensile strength than the rolled section from which it is drawn, so that if we want to make it easily bendable, and take that hardness out of it, we anneal the wire. It then becomes mild again, as in the original rolled section Here, in passing, reference may be made to the fact that some of the mills rolling steel bars for reinforced concrete expend great power to force the smaller sections through the rolls when the steel is cooling very appreciably, and the final rolling, in the smaller sections, therefore entails doing a certain amount of cold work by causing plastic deformation at a lower temperature. This hardening is more noticeable on the outer skin of the bars. Thus it will be found that big bars which have been rolled from the billet in a shorter time, and therefore at a higher temperature, though from the self-same melt of steel will have a lower tensile strength than the smaller bars, and further that if the smaller bars are put in a lathe and turned down to afford test pieces they often show a lower strength per square inch than the full section will do from which they have been turned by reason of the hard skin having been turned off leaving the soft, mild steel.

The fact that structural materials of this kind can be changed in nature by heat treatment suggests the probability of changes by other physical conditions. Professor Chunder Bose has demonstrated that inanimate materials respond to physical and chemical stimuli, just in the same way as living matter, and that material can be fatigued—as one might say —by subjection to heat and electrical treatment, as also poisoned by chemicals. Though the subject of the fatigue of structural materials has exercised a good deal of thought and a lot of experiments have been made, yet we do not know so very much about the real causes of fatigue, and remedies therefor. In the case of steel, it has been pointed out that heat treatment can cause something in the nature of fatigue if that means a change in the structure of a material. Fortunately, in order thus to reduce the strength of cold worked steel the temperature has to be raised very considerably. Experiments do not seem to indicate that ordinary chemical, electrical, or other physical influences reduce the strength either. The so-called fatigue tests of

Wöhler and others have given results which are far too similar, in the author's opinion, to those given by shock or impact tests, indicating that there may have been some little eccentricity which induced bending in the shafts having loads suspended thereon, so as to cause amplification of vibration at critical high speeds, and produced stresses in excess of the yield point of the material. Naturally, if permanent set can continue to be produced in that way the section must gradually be reduced, and undergo fracture.

There are many instances in practice that would appear to indicate that ordinary vibration due to mild shocks of impact do not cause failure. For example, pianoforte wires subjected to very high tensile stresses will endure for many years, only failing finally when tightened up excessively by tuning, or, if deteriorated by rust.

Engineers have found out by experience the advisability of using mild low-carbon steel in structures as against brittle high-carbon steels. Mild steel, as a ductile material, has the virtue of not losing strength in either tension or compression, by undergoing plastic deformation. Perhaps structural engineers, as regards so-called brittle materials, are more familiar with those that have low tensile strength like concrete, stone and brick. This is not a necessary feature of a brittle material as, for example, high carbon steel which may have a high tensile strength, and yet be brittle. The brittleness merely indicates that the material is not malleable, or appreciably deformable, without loss of strength in tension. We are afraid of brittle materials in tension and in bending because the bending produces tension, but we are not so afraid of them in compression. With ordinary building materials, such as concrete, brick and stone, the general use is for compressive resistance, and we are not afraid of a good deal of deformation. With materials that are comparatively weak in tension it is difficult to observe, in tensile tests, either a yield point or how much plastic deformation can be undergone without causing failure. Probably the way in which to get at this best is to make long time compression tests on such materials as concrete and stone. The author has observed in making compression tests on concrete, as no doubt others have, that if one applies the loading of a fairly rapid rate a higher result may be obtained than if the load be applied slowly, and has further noted that if a load near the ultimate strength, determined as usually, be kept on a specimen for, say, an hour or two, rupture will take place at the lower load.

In Johnson's " Materials of Construction," 4th Edition

a number of diagrams are given of tensile stress-strain curves for steel with different amounts of carbon therein, together with some for nickel steel. The data given appears to indicate that in tool steel and the like there is still a yield point, but that it approaches closely to the ultimate strength. In nickel steel considerable plastic deformation can occur, but rupture takes place with very little increase in the stress beyond the yield-point. Up to the yield-point there is very little plastic deformation. A high yield-point, and high ultimate strength can therefore be obtained without sacrifice of ductility, but usually high carbon steels approach the nature of cast-iron, in which there is very little ductility. There may be a yield-point if close enough observation can be given, but it will be near the ultimate, and, in all probability, the plastic deformation that can occur will be very little.

Little or nothing is known about the effects of fatigue on brittle granular materials when they are subjected to compression. In all probability they are not hardened or increased in strength by plastic deformation. On the other hand, they do not appear to be reduced in strength thereby. There is no marked yield-point. As has been pointed out in the author's former paper on "Shear," the compression strengths of such brittle granular materials depend in part upon the cohesive or tensile resistance so that stress-strain diagrams for compression on concrete and stone, may enlighten us somewhat as regards what might be expected if we could carry out careful tensile tests. They indicate that a good deal of plastic deformation is being undergone all the time, and that if we unload and re-load several times, we render the material more elastic in its deformation, so that in a sense an elastic limit—and almost a yield point—is created by re-loading. We can approach with each re-loading an artificially created yield point nearer and nearer to the ultimate resistance in a very similar way to what can be done with mild steel by stressing beyond the normal or original yield point as has been explained above.

Though engineers may be afraid of brittle steel in rivetted steel structures where, owing to deflection under load uncalculated secondary stresses are put into the rigid joints, we are accustomed to using such materials for other engineering purposes. Where brittle materials have low tensile strengths compared with their compressive resistances, such as cast iron, concrete and stone, we fear to use them in tension, yet we are familiar with their use in compression. Their ability to undergo considerable plastic

deformation must be a reason for their practical utility. Certainly by re-loading, we see that it may be possible to render them as brittle in compression as tool steel. If we could use materials that had been re-loaded many times, so as to make them elastic, but having elastic limits near to their ultimate strengths, we should be just as afraid of using them in positions where secondary stresses might be produced as we are in the case of brittle steel in rivetted structures. When, however, we use them in brick and stone buildings and in reinforced concrete structures, they are built into the work in a comparatively unstrained state, and are, therefore, able to undergo on loading a great deal of plastic deformation.

The formulæ which are ordinarily put forward as applying within the limits of elasticity, may give fairly accurate data for structural steel work, and they provide practically the only method of calculating slender pillars and continuous construction though that does not mean that they even closely approximate to really accurate results. The settlement of supports and plastic deformation must make appreciable divergencies.

The author is familiar with the methods of calculation in which the slopes of deflected beams and pillars are employed to derive equations for mathematically determining the bending moments and shearing forces in rigidly connected or monolithic structural members of elastic materials, as also with the methods of employing the theory of least work such as admirably set out by Castigliano, of whose book Mr. Ewart S. Andrews has recently published an English translation. The theorem of least work is indeed only a variation of the principle of minimum energy, which is adopted for the solution of unknowns in equations relating to all sorts of physical and chemical problems. The author uses such methods for approximate computations of continuous or monolithic construction; but such methods are only adopted for want of better. The mathematical work that has been done has been of very great value to the engineering profession; but engineers in computing practical structures have to remember that the materials of which they are composed are not ideally elastic bodies.

The author is enthusiastically in favour of scientific enquiry of any and every kind,, and believes that out of the investigations of structural materials, both chemical and physical, we derive immense benefit, but engineers have to proceed in advance of complete knowledge of scientific laws and therefore must use to the best of their ability the scientific

information and mathematical tools placed at their disposal to effect practical but naturally approximate solutions of the problems before them.

BENDING OF HOMOGENEOUS BEAMS.

In the ordinary theory as regards stresses in beams as set out in the usual text books on the theory of structures, we find statements about the stresses in the fibres of a beam bent by transverse loading varying as the distance from the neutral axis, the said neutral axis being the position in the beam above which the fibres are shortened by compression, and those below extended by tension.

Frequently, also, statements are made that plane sections remain plane after bending. The author's former paper on "Shear" demonstrated that, in practice, this were not so, and indeed that the assumption never is warranted. It may be an interesting graphical illustration, but seeing that it is foreign to the truth the wisdom of putting it forward to the student is doubtful. It was pointed out in the former paper that shearing stress will cause the fibres of a beam to be drifted past each other, as it were, so that all planes will be warped. Again, if a steel ball were embedded in a lump of jelly, and the jelly were bent, it is evident from Figure 3 that the planes which are indicated in illustration by the lines A B and C D respectively would no longer be planes. They would be curved surfaces consisting of sorts of planes with humps or cavities therein There is, of course, no necessity to refer to any planes remaining plane. All we have to do is to assume some variation of strain and of stress across the section. To take our illustration of the jelly, we can assume that the strain in the jelly varies from the neutral axis outwards in some uniform manner, though the strain in the steel ball will naturally be very different from that of the jelly adjoining it. We see the same statement about plane sections remaining plane after bending in books on reinforced concrete. This is particularly aggravating when the authors go on and point out that concrete cannot be depended on in tension, as it will probably crack, and, indeed, cracks can be expected in reinforced concrete. If a crack occurs in the concrete, and not in the steel bar, obviously the plane section cannot remain plane after bending. The concrete has either slipped past the bar or the bar past the concrete.

If a material is perfectly elastic and undergoes equal strain for equal stress in tension or compression, it is evident

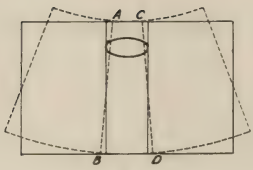


Fig. 3

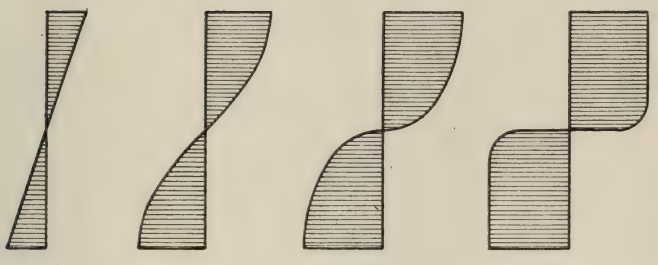


Fig. 4.

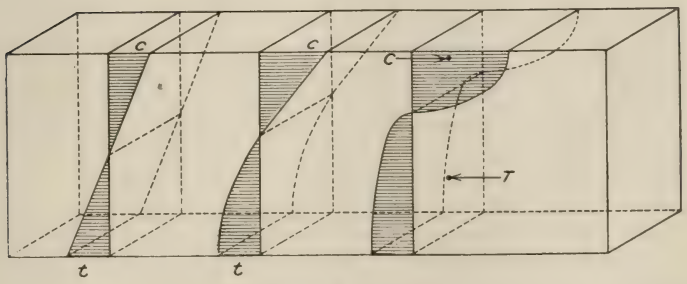


Fig. 5.

that in a symmetrical section the neutral axis will lie in the middle. There is a neat graphical demonstration for the determination of the neutral axis in unsymmetrical sections in a book which is too little known to structural engineers, namely, "Elementary Applied Mechanics," by Alexander and Thompson, pupils of Rankine. For students unfamiliar with the calculus their demonstration of the moment of inertia, and the section modulus of a figure is one of the best. With that one has no quarrel because the authors clearly point out that it only applies on the assumption of a uniformly varying stress. We can, of course, modify such a method to suit materials having different moduli of elasticity in tension and compression, and even variable moduli, but artifice is necessary.

Another familiar mode of calculation is to employ the theorem of least work, but it still requires us to assume the distribution of strain and stress just as before. That method has been applied to reinforced concrete beams, based upon the same assumptions as the ordinary theory. The demonstration is free from the error about plane sections remaining plane, yet it depends upon the truth of the assumptions.

It is a well known fact remarked upon in many books on the " strength of materials " and " theory of structures" that ordinary rectangular sections of homogeneous material when subjected to bending develop moments of resistance at ultimate loading that are greatly in excess of what would be expected by multiplying the section modulus (derived from the moment of inertia) by the maximum tensile stress that the material is able to sustain. Or alternatively if the ultimate moment of resistance be divided by the section modulus a maximum fibre stress is indicated far in excess of what can be ascertained by pure tensile tests. Sir Benjamin Baker remarked upon this in his book published in 1870, and gave the value for cast iron rectangles as being apparently 2.25 times the ordinary tensile stress, or put in another way the ultimate resistance moment was $2\frac{1}{4}$ times as much as was expected from theory. For wrought iron the corresponding value that he obtained was 1.57. Another striking divergence is in regard to what has been termed the structural paradox, namely, the statement that " the whole is weaker than the part " as applicable to a triangular beam section If one sharp edge be blunted it is demonstrable that the moment of resistance in respect to the stress on that edge will be greater for the blunt section, yet as a matter of fact experiment did not

justify the paradoxical statement. The reason in all these cases is similar, namely, that in the final stages of loading the outer fibres become overstrained, with the result that the material begins to deform plastically, thus throwing gradually more and more on the inner fibres.

The successive stages of stress distribution for a symmetrical section of a material with equal tensile and compressive functions are shown in Fig. 4, and for materials having a tensile strength less than the compressive in Fig. 5 If we assume the ultimate stress figure to be something in the nature of a quarter ellipse we can determine the centres of gravity of the stresses—i.e., the centres of pressure and tension—the arm between and the amounts of total tension and total compression.

Now let us consider a simple little demonstration of the fundamental mechanics of a beam. Fig. 6 illustrates a cantilever projecting from a wall and carrying a weight $\frac{W}{2}$ at the end, distant a length $\frac{l}{2}$ from the support. Fig. 8 shows a beam carrying a load W about the middle of the span l. The reaction at each end is obviously $\frac{W}{2}$. If the figure be reversed it will be evident that the conditions of each half are similar to that of the cantilever, bending taking place about the centre. The load or reaction amounting to $\frac{W}{2}$ is in each case acting with a leverage $\frac{l}{2}$. The weight multiplied by the leverage gives a moment tending to cause bending, known generally therefore as the bending moment, equal in this case to $\frac{W}{2} \times \frac{l}{2} = \frac{Wl}{4}$. Reverting to the simple case of the cantilever shown in Fig. 6, let us imagine that the projecting portion beyond the dotted line is separated from the support as shown in Fig. 7. This body is subjected to the application of a force at one end of $\frac{W}{2}$, which tends to cause motion of translation. As the body is in equilibrium this force of $\frac{W}{2}$ must evidently be resisted by some force applied at the cut section where it was attached to the support. This equal and opposing force is the shear, symbolised by the letter S, as indicated in Fig. 7. But

the application of two equal and opposite parallel forces upon a body tends to cause rotation. This rotation must evidently be resisted by a force or forces applied from the adjoining piece of the beam at the cut section. One force, however, is insufficient to produce equilibrium, because alone it would merely cause motion of translation, which could only be stopped by the application of an equal and opposite force. These forces are indicated in the diagram by the letters T and C, representing tension and compression. They in turn form a couple, which tend to cause rotation, the direction of rotation, however, being opposite to that of the first couple, which consist of the load and the shear. Taking moments about any point the first couple give an amount of $\frac{W}{2} \times \frac{l}{2}$. This is the bending moment before referred to. As regards the other couple of forces of C and T, representing total compression and total tension, acting at a distance apart equal to a, the leverage arm of the couple, taking moments for this couple of equal and opposite forces about any point (similar to what we have done for the first couple), we find that the moment is Ta or Ca, these being equal, because $C = T$. This moment resists the moment of the other forces, and is known as the resistance moment. In short, the resistance moment is equal to the bending moment, and the total compression equals the total tension.

In the case of materials that are weaker in tension than in compression, at first the neutral axis will lie at the centre of the section, as at the left of Fig. 5. As the weak tension side is overstrained and the inner fibres become stressed more nearly to the outer fibres, the neutral axis will move upwards so as to make the volumes of the tension and compression stress figures equal. If, of course, the conditions were reversed, by the material being stronger in tension than in compression, the neutral axis would have to travel downwards by the same argument. However, in the present case with cast-iron or concrete, as the neutral axis travels upwards the outer fibre stress in compression becomes gradually increased, while more and more is brought into play in tension, until finally the area in compression becomes so small that the ultimate compressive resistance of the material is reached when by plastic deformation the inner fibres become more uniformly stressed. By such argument both ultimate stresses are reached at the same time. Assuming elliptical stress distribution as before we can calculate the ultimate resistance moments if we know the ultimate tensile and compressive stresses.

Referring to figure 5, let

$$f\,max = N.f\,min$$

$$\because\ C = T$$

$$\therefore\ \cdot 7854b.f\,max.n = \cdot 7854b.f\,min\,(d-n)$$

Also $f\,max.n = f\,min\,(d-n)$

$$\therefore\ n = \frac{f\,min}{f\,max + f\,min}\,d$$

$$a = d - \cdot 4244\,n - \cdot 4244\,(d-n) = .5756\,d$$

$$R_u = \cdot 7854 \times \cdot 5756 \cdot \frac{N.f\,min^2}{f\,min\,(N+1)} \cdot bd^2$$

$$= \cdot 452\,\frac{N.f\,min}{(N+1)} \cdot bd^2$$

Ordinary Elastic $R_e = f\,min\,\frac{bd^2}{6} = \cdot 166 f\,min.\,bd^2$

For $N = 4$ $\quad R_u/R_e = 2 \cdot 17$

For $N = 1 \cdot 3$ $\quad R_u/R_e = 1 \cdot 53$

On the basis that the tensile strength of cast iron is roughly about $\frac{1}{4}$ of the compressive resistance we obtain the ratio of ultimate to elastic resistance moments as $2 \cdot 17$ which is near the value $2 \cdot 25$ given above, while for wrought iron with tensile strength $1 \cdot 3$ times the compressive strength we get $1 \cdot 53$ as against the value $1 \cdot 57$ given above. This is close agreement with experiment.

Such a theory is extremely simple if we once assume the shape of the stress figure at the ultimate. The only thing we need is a fairly close approximation to the true stress distribution. If much squeezing goes on by plastic deformation of the fibres the stress figure may become almost a rectangle or something like the right-hand diagram of Fig. 4. For concrete it has been suggested by various experimentalists that the stress-strain curves for concrete in compression indicate that a parabola would be justified, merely having regard to the varying modulus of elasticity of the concrete, but the author would prefer to adopt an ellipse as being justified by the known squeezing of the outer fibres; that gives a figure more approaching the rectangle than does the parabola, while, on the other hand, it must be fairly evident that a complete rectangle cannot be justified. The ellipse is a figure that is somewhere between the two, with simple known properties.

Referring to Fig. 5, as applicable to concrete or cast-iron, requiring merely a modification to suit the particular properties of each material, we see that at first when the material is fairly elastic the neutral axis will be at the centroid of the cross-section, because the strain will vary directly as the distance from such axis, and likewise the stress. The material being weaker in tension than in compression the ultimate tensile strength will evidently be reached at an early stage of continued loading. As soon as that stress is reached, stretching must occur, i.e., plastic deformation. As a result the neutral axis will move upwards, the strain will no longer be directly proportional to the distance from such axis, and the stress will become somewhat as sketched in the second part of the diagram. The succeeding stage is likewise indicated. Such considerations would indicate that unless the material has some power of recovery, plastic deformation under repeated loading will become more and more until no further give is possible. Seeing that the inner fibres are not so greatly strained as the outer fibres, the deformation in them is probably more elastic, or, to put it another way, the plastic deformation in such fibres will be less. The effect of this will be that on removal of the load these inner fibres will tend to recover themselves more than the outer fibres, and will help to push the latter back into their original condition. This can be noticed in certain materials by a gradual creeping back of deflected beams, so that they become straighter after a period of rest than they were on first removal of the loads after overstrain.

By repeated plastic deformation materials, as we have seen, become nearly perfectly elastic, but at the same time brittle. Each time of reloading, therefore, the neutral axis will start at the middle, and subsequently the tension side will be overstrained and more plastic deformation produced. The result must be eventual cracking and rupture on the tension side. This is somewhat allied to the effect of failure by shocks or by fatigue. Indeed, being due to repeated loading it might properly be termed fatigue, though the effects are similar to overstrain by impact, though the overstrain is due to an entirely different cause. As to whether fatigue in structural materials is really of this nature is a question that cannot be definitely answered. It somewhat depends upon what we are going to term fatigue. If we are going to use the word in an analogous sense to that of becoming tired it would be right to attribute the fatigue to overstrain. There is, however, another aspect that physical deterioration or fatigue might be the cause of overstrain, that is to say, that

a force that would not cause overstrain at first might cause overstrain if the structure of a material were altered by other influences outside of the action of the force itself, as, for instance, heat, or electrical treatment, or chemical change by some delayed re-action between the constituents of the material.

(To be continued).

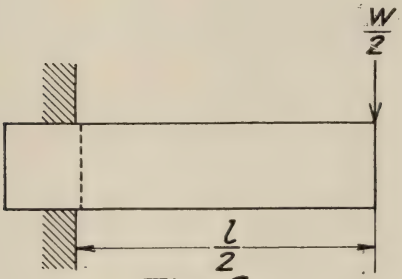


Fig.6

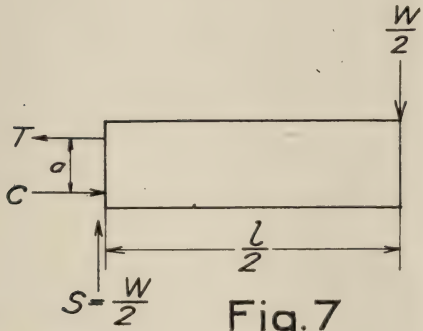


Fig.7

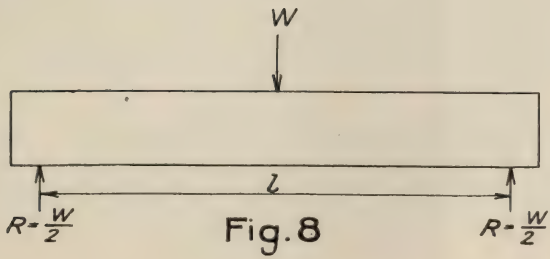


Fig.8

NEW BUILDINGS IN LONDON IN WHICH CONCRETE, ETC., IS BEING USED.

BERMONDSEY. L.C.C. Webb Street Mixed Elementary School. Reinforced concrete floors. Architects: L.C.C. Builders: Messrs. L. H. & R. Roberts, 144, Lower Clapton Road, E.5.

Garage for Messrs. Pickfords, Ltd., in Tower Bridge Road. Steel framework fireproofed with concrete. Builder: H. Kent, Beacon Road, Hither Green, S.E.13.

WEST CENTRAL. Extension to Lyons' Corner House, with reinforced concrete floors, by Messrs. F. Bradford & Co., 23, Coopersdale Road, E.9.

ISLEWORTH. Community of Gumley House, Isleworth, additions to College Buildings. All the flats are in reinforced concrete covered with asphalte. Contractor: W. Lacey, Staines Road, Hounslow.

CHARLTON. Extension to L.C.C. Tramways Depot in Woolwich Road, buildings with steel framework fireproofed with brick and concrete; reinforced concrete floors. Contractors: Messrs. P. & W. Anderson, Ltd., 199, Piccadilly, W.1.

KILBURN. Hall for the Catholic Church in Quex Road. Foundations of reinforced concrete blocks, made on the site. Reinforced concrete floors. Contractor: W. Lacey, Staines Road, Hounslow.

BATTERSEA. Southampton Wharf Grain Store for Messrs. Garton, Sons & Co., is being constructed almost entirely in concrete. Architects: Messrs. Trant, Brown & Brightiff, High Road, Kilburn, N.W.6. Contractors: Messrs. Holliday & Greenwood, of Battersea.

WILLESDEN JUNCTION. Factory for the Chesebrough Manufacturing Co., to be built in steel and reinforced concrete. Designers: Messrs. Johnson's Reinforced Concrete Engineering Co., Ltd., 1, Basinghall Street, E.C.2.

Information regarding the use of reinforced concrete in new buildings, together with the names and addresses of the Architects and Builders, will be welcomed and should be sent to the Secretary of the Concrete Institute.

PROCEEDINGS.

The 109th Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, February 23, 1922, at 7.30 p.m.

The chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous meeting as published in the *Journal*, were taken as read, and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association:—

Members.

ALLSEBROOK, VIVIAN, Messrs. Tidmans', Ltd., Wisbech; BRINE, RICHARD HAROLD, M.S.A., 22, Old Square, Lincoln's Inn, W.C.2; CANE, Mrs. MAY, 100, Victoria Street, S.W.1; COX, ARTHUR STANLEY, F.S.I., M.S.A., Lic.R.I.B.A., 16, Queen Victoria Street, Reading; HICKSON, CLIFFORD, A.R.I.B.A., M.S.A., 1, St. Peter's Street, Huddersfield; HOULDSWORTH, AUSTIN FRANK, M.S.A., P.A.S.I., 20, Savile Street, Hull; HOUSTON, JAMES, A.R.I.B.A., M.S.A., "Glenlogan." Kilbirnie, Ayrshire; HOWELLS, DAVID JOHN, A.R.I.B.A., M.S.A., Clase Road, Morriston, Swansea; LISTER, ALFRED HENRY, M.S.A., 18, Adam Street, Strand, W.C.2; MAXWELL, HERBERT PERCY, M.S.A., Market Place, Pocklington, E. Yorks; ROBERTS, DAVID HENRY, M.S.A., Park Street, Wombwell; ROBERTS, J. WILLIAMS, M.S.A., M.R.San.I., 36, High Street, Pwllheli; REES, FREDERICK WILLIAM, M.S.A., 44, Park Lane, Croydon; SAINSBURY, FREDERICK GEORGE, M.S.A., 154, Friar Street, Reading; SHORTHALL, JOHN FRANCIS, F.S.I., M.S.E., Office of Public Works, Dublin; WEEKS, ALFRED HENRY, F.S.I., M.S.A., 46, Fairfield Avenue, Cardiff.

Associate-Member.

O'HANLON HUGHES, JAMES, M.S.A., 46, Maitland Park Road, Haverstock Hill, N.W.3.

Resignations.

The Secretary announced the acceptance by the Council of the resignations of H. HASTINGS and G. E. L. POULDEN, Members. ERRATA.—A printer's error under this heading in the last issue of the *Journal* should read:—The Secretary announced the acceptance by the Council of the resignations of PERCIVAL STEVENS and F. E. L. HARRIS, members.

Death.

Great regret was expressed on the announcement of the death of E. H. HUDSON; member, of Jalwa, India.

Mr. H. KEMPTON DYSON (member of Council) then read a paper entitled, "WHAT IS THE USE OF THE MODULAR RATIO?" and a hearty vote of thanks having been accorded, a lengthy discussion took place (reported elsewhere).

Mr. DYSON replied to the questions raised in the discussion, and the proceedings terminated.

OFFICERS AND COUNCIL, SESSION 1922-1923.

PRESIDENT.

E. FIANDER ETCHELLS, A.M.Inst.C.E., A.M.I.Mech.E., Hon. A.R.I.B.A., &c.

VICE-PRESIDENTS.

J. ERNEST FRANCK, F.R.I.B.A.
Dr. J. S. OWENS, B.A., M.D., F.R.G.S., A.M.Inst.C.E., &c.
H. J. DEANE, M.Inst.C.E.
Major JAMES PETRIE, O.B.E.
C. F. MARSH, M.Inst.C.E.

PAST PRESIDENTS.

The Rt. Hon. the EARL OF PLYMOUTH, P.C., G.B.E., C.B.
Sir HENRY TANNER, C.B., I.S.O., F.R.I.B.A.
Professor HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., F.S.I., F.S.Arc., F.R.San.I., &c.
F. E. WENTWORTH-SHEILDS, O.B.E., M.Inst.C.E.
H. D. SEARLES-WOOD, V-P.R.I.B.A., F.R.San.I.

COUNCIL.

E. S. ANDREWS, B.Sc.
H. K. G. BAMBER, J.P., A.M.Inst.C.E.
P. J. BLACK.
H. F. BLADEN.
W. E. A. BROWN, A.R.I.B.A.
D. B. BUTLER A.M.Inst.C.E.
W. C. COCKING.
J. S. E. DEVESIAN, M.I.Mech.E.
H. KEMPTON DYSON.
Dr. OSCAR FABER, O.B.E., A.M.Inst.C.E.
W. A. GREEN, M.A., B.Sc.
E. L. HALL.
B. L. HURST, M.Inst.C.E., M.I.Mech.E.
W. J. H. LEVERTON, F.S.Arc.
Prof. E. R. MATTHEWS, A.M.Inst.C.E.
A. E. MESTON, Lic.R.I.B.A.
F. PURTON, F.S.E.
SIR CHARLES T. RUTHEN, O.B.E., F.R.I.B.A., F.S.Arc.
A. ALBAN H. SCOTT, F.S.Arc.
ARCHIBALD SCOTT, M.B.E., A.R.I.B.A.
R. H. HARRY STANGER, A.M.Inst.C.E.
H. E. STEINBERG, A.M.Inst.C.E.
R. W. VAWDREY, A.M.Inst.C.E.
G. C. WORKMAN, M.S.E.
M. E. YEATMAN, M.A., M.Inst.C.E.

SECRETARY.

Captain M. G. KIDDY, F.I.S.A., M.J.I.

SOLICITORS.

FIELD, ROSCOE & CO., 36, Lincoln's Inn Fields, W.C. 2.

AUDITORS.

MONKHOUSE, STONEHAM & CO., Salisbury House, E.C. 2.

BANKERS.

LONDON, COUNTY, WESTMINSTER & PARRS BANK, LTD., Victoria Branch, S.W.

THE CONCRETE INSTITUTE
PUBLICATIONS.

"Transactions and Notes."—

Vol. IV., Pts. 3 and 4	(Each)	**2s. 9d.**
,, V., Pt. 1		**5s. 6d.**
Pt. 2		**2s. 9d.**
Pts. 3 to 6 (inclusive)		**11s. 0d.**
Vols. VI. (Pts. 1 and 2), VII. and VIII.		**5s. 3d.**
,, IX.		**5s. 9d.**

"Standard Notation for Engineering Formulæ" ... ... **6s. 3d.**

In this Report the Science Committee provide a system of standard symbols in order that time and consequently money may be saved in writing down the calculations which all classes of engineers are making daily in following their professions. The idea is to provide means whereby mathematical equations can be typed more easily, and printed more cheaply, and also to lighten the student's task by providing a set of self-explanatory symbols which will become imprinted upon the memory almost automatically, and be retained there without effort.

"Report of Joint Committee on Loads on Highway Bridges" **2s. 8d.**

Effect of speed increasing virtual load; Table of standard loading for bridges of three classes; diagram of typical loads; tractive efforts on various road surfaces; notes on heaviest loads occurring in different districts; bending moment diagrams for standard loading; shearing forces; calculated example; effect of width of tyre and diameter of wheel; maximum axle loads allowed in various districts: Acts and Orders relating to motor traffic; scale diagrams of heavy vehicles with Table of loads and dimensions; determination of bending moments and shearing forces for beams subjected to moving loads.

"Recommendations to Inspectors, Clerks-of-Works and Foremen concerning the execution of Reinforced Concrete Works" ... **6½d.**

"Standard method of measurement of Reinforced Concrete in Building construction," as adopted by the Concrete Institute and the Quantity Surveyors' Association. ... **1s. 1d.**

This pamphlet contains a full description of the order in which the work should be taken, the items which should be measured and grouped separately, and the mode of measurement in each case for *in situ* and *pre-cast* work.

"A Standard Specification for Reinforced Concrete Work" **1s. 7d.**

Reports to the Scientific and Industrial Research Department on the Research Work on Concrete carried out under the direction of the Concrete Institute (1917-1919) ... ... **1s. 7d.**

Post Free (Inland).

The Year Book.

Publication of the Year Book has been postponed for a few weeks only, pending certain decisions to be made by the Council. Meanwhile members are asked to communicate any changes in rank, style, or address, to the Secretary of the Institute, at the earliest date.

Papers for Session 1922-23.

Members desirous of reading papers before the Institute during the Session 1922-1923 are invited to communicate with the Secretary of the Institute, at 296, Vauxhall Bridge Road, S.W.1. The title of the paper should be stated, a summary of the subject matter should be enclosed, and a suitable date should be intimated.

Papers should not exceed 10,000 words in length for printing, and are subject to approval by the Literature Standing Committee.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s. 0d.
		Remainder ...	£2	2s. 0d.
ASSOCIATES	,, ...	... ...	£2	2s. 0d.
ASSOCIATE-MEMBERS	,, ...	... ...	£1	11s. 6d.
GRADUATES / STUDENTS	,, ...	... ...		10s. 6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

CONCRETE INSTITUTE MASONIC LODGE.

Some members of the Institute decided, at a meeting held on March 9, 1922, to present a petition to Grand Lodge for the founding of a Concrete Institute Lodge. Members interested should communicate with Captain M. G. Kiddy, at 296, Vauxhall Bridge Road, S.W.1.

EXAMINATION IN MAY.

Examinations, Part I. for Graduateship and Part II. for Associate-Membership, will be held in London and at other centres as may be arranged, on Thursday and Friday, May 18 and 19.

A Syllabus and full particulars may be obtained on application to the Secretary of the Concrete Institute, 296, Vauxhall Bridge Road, S.W.1.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *MAY, 1922.* **No. 5.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

Papers for Session 1922-23.

Members desirous of reading papers before the Institute during the Session 1922-1923 are invited to communicate with the Secretary of the Institute, at 296, Vauxhall Bridge Road, S.W.1. The title of the paper should be stated, a summary of the subject matter should be enclosed, and a suitable date should be intimated.

Papers should not exceed 10,000 words in length for printing, and are subject to approval by the Literature Standing Committee.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s. 0d.
		Remainder ...	£2	2s. 0d.
ASSOCIATES	,, ...		£2	2s. 0d.
ASSOCIATE-MEMBERS	,, ...		£1	11s. 6d.
GRADUATES, STUDENTS	,, ...			10s. 6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

WHAT IS THE USE OF THE MODULAR RATIO?

PART II.

By H. KEMPTON DYSON.

REINFORCED CONCRETE BEAMS.

Coming now to the consideration of reinforced concrete beams, if we turn back to the early pioneers we find first of all that their work was done with next to no calculation; slabs and beams were tested to destruction, and in the light of such experiments buildings and parts of buildings were constructed, the sizes being roughly proportioned from the data thus obtained. The first attempt to formulate some method of calculations that is known to the author was that given in a book printed at Chiswick in 1877 for private circulation by Thaddeus Hyatt, the inventor of pavement lights, who came over from New York to establish a business therein in England, and had a number of experiments on reinforced concrete beams conducted by David Kirkaldy. The method of calculations put forward in the book was a simple comparison with iron joists embedded in concrete. It was pointed out that the upper flange of a steel joist was in compression, and the lower in tension, and that the web and compression flange might be omitted, the purpose served thereby being fulfilled by the concrete. All the concrete above the middle of the steel joists was taken as being in compression. The ultimate compressive resistance of the concrete was taken as 2,000 lbs. per square inch, but this was reduced to a mean stress of 1,000 lbs. per square inch, which looks like a triangular stress distribution, but the centre of gravity is taken halfway, thus suggesting rectangular stress distribution. Mr. Hyatt acknowledged his indebtedness to Mr. Thos. Rickett, of Birmingham, formerly, as he says, in the employ of the London and North-Western Railway. Probably Mr. Rickett was responsible for the method of calculations, which are merely sketched in the book. It is interesting to note that Mr. Rickett and Mr. Hyatt did not lose sight of the possibility that the neutral axis might become displaced by reason of the different deformations of concrete and steel under a similar stress, because they record an experiment that they made to ascertain the amount of deformation, but apparently through the roughness of the apparatus they were not able to get any difference between wrought iron and cement, so assumed the same for concrete (the book states) " the effect of which in the compound beam is to keep the neutral axis

at the central line of the beam, the entire tensional strain being thus thrown on the metal tie."

A rather similar mode of calculations was later recorded in a catalogue published by Wayss and Freitag, the contractors who first exploited the Monier patents in Austria and Germany. When the pioneers of reinforced concrete construction in Germany, Belgium and France began to get well under way and to carry out extensive structures in the material, some discussion took place amongst continental engineers as to the proper mode of calculation. Mons. Edmond Coignet, together with his engineer Mons. N. de Tedesco, it has been stated, were the first to suggest introducing the modulus of elasticity of the concrete, and of the steel into the calculations. Then Monsieur Considère and others elaborated the theory, and at first in the light of some experiments of his, the concrete on the tension side of a rectangular beam of reinforced concrete was taken into account. Then when it was found that in practice the concrete was cracked on the tension side the tensile resistance of that material was omitted, and the ordinary current theory was arrived at, to which the American Professors Hatt, Talbot, and Withey, and the German Professors Mörsch and Bach made notable contributions, from whose experiments a justifying co-efficient was chosen to make the formulæ somewhat fit the experiments, the said co-efficient being now termed the modular ratio, seeing that it purported to be derived from the moduli of steel and concrete, the two elements of the combination.

For many years past the author has expressed dissatisfaction with the customary mode of calculating stresses in reinforced concrete members transversely loaded, because some of the assumptions upon which the mathematical treatment is based are illogical, while others are not in agreement with fact. In the paper on shear, to which reference has been made above, it was pointed out how the concrete was ignored in tension because of the stated possibility of cracks extending from the convex side of a so-called "beam," and yet certain modes of calculation had been put forward as a means of determining the shearing stresses whereby horizontal shear was required to be transmitted from section to section, forgetting that it could not pass across a crack. In short, the demonstrations were inconsistent. But the trouble was deeper rooted than that. It was not merely the case that the method of determining shear stresses referred to was wrong, but the fundamental assumption about the stresses being directly proportional to the distance from the neutral axis was wrong—again

because of the cracks. A reinforced concrete member subjected to transverse loading may, if uncracked unitially, start by the strain being directly proportional to the distance from the neutral axis, which will be located not at the ordinary centroid of a homogeneous section, but in a position corrected for the steel having a different modulus from the concrete. But though at first a reinforced concrete member may act truly as a beam, the neutral axis of which, allowing for the reinforcement, starts at approximately 54% of the effective depth from the compressed edge, yet as, in order to obtain economy, we naturally reckon upon putting a fairly high tensile stress into the steel under working loads, the result will be that with quite a moderate stress in the steel the concrete will be cracked on the convex side. These cracks are minute as a rule, so that when air penetrates to the steel no corrosion will take place, by reason of the presence of the alkaline lime compounds of the cement. These cracks are reduced in size and in number by making members fairly deep. After all, the cracks are due in a sense to curvature in bending, and as the deflection varies directly as the cube of the depth, it is easy to see that a small increase in depth will make a great difference in the deflection and the radius of curvature. In properly designed reinforced concrete, cracks are very small and may not be noticeable to the naked eye, nor to the casual observer, so that such structural members may appear to be homogeneous units. The fact that cracks do occur, however, renders the structures really compound ones, in which the concrete and steel act as separate elements, though connected together. Thus, in reality, such a member is not a beam—it is more in the nature of an arch or elemental frame.

Figs. 9 and 10 illustrate the sort of simple frames that result from concentrated or point loads on so-called beams of reinforced concrete, the dotted lines representing the compression struts produced within the concrete. The drawings also illustrate the probable form of the minute cracks that would be formed in the underside about the middle of a beam by the stretching of the steel.

Figs. 11 and 12 are similar diagrams, but in relation to two concentrated loads upon a single beam. When the load is evenly distributed, the compression-strut action results in producing a curved formation analogous to a jack arch with a tie-rod for resisting the thrust in the arch (see Fig. 13).

When web members are introduced, the construction takes the form of a trussed frame, as indicated in Figs. 14, 15 and 16; the first being similar to an N-girder, and the second

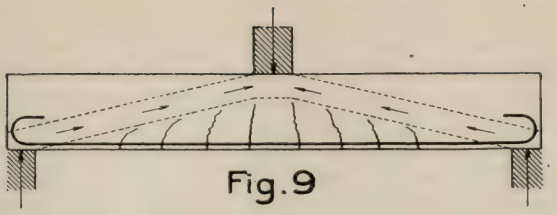


Fig.9

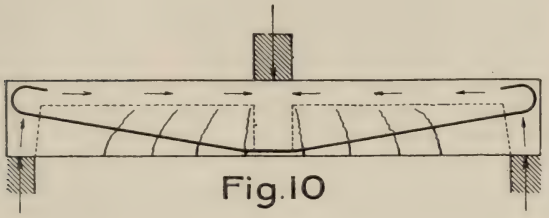


Fig.10

Fig.11

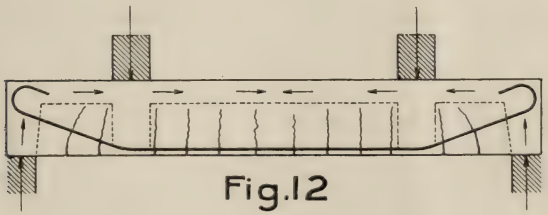


Fig.12

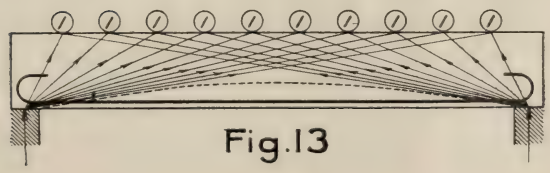


Fig.13

and third to a Warren girder. By super-imposition of Figs. 15 and 16, we get the combination (Fig. 17) which is very often employed in practice. There may also exist at the same time frame or arch action similar to that indicated in Figs. 9 to 13. The combination of vertical with inclined members is frequently adopted to still further complicate the frame and truss action, while the uncracked portions often act partly as homogeneous beams. Thus an attempt to make an accurate detailed analysis of the stresses at stages preliminary to rupture is impracticable. There are, however, certain fundamental relations in beam action that enable the breaking strength to be calculated fairly closely.

From the foregoing consideration the conclusion is drawn that we are not very sure as to how a so-called " beam " of reinforced concrete is really going to act: it may be a sort of arch or a simple frame, or a truss. When, indeed, it comes to building a slab to rest on its edges there may be developed a sort of flat dome or a cone or a pyramid within the thickness.

As to how much of the concrete is cracked throughout the length of the member cannot be determined with any accuracy. When the concrete cracks, the steel reinforcement will slip through. A series of elaborate tests carried out in recent years have shown a gradually sliding of the bars in the concrete, as the loads have increased, until most of the pull is taken by the hooks or other anchorages at the ends. We have noted that for the first time of loading the action is altogether different from what it is when cracking has occurred. Towards the ends, therefore, of a " beam " there may be uncracked sections in which the distribution of stress would be very different from that in a frame. We see, therefore, that we cannot know at all precisely the actual distribution of the deformation or strain throughout the member; consequently we have no means of determining except in the roughest manner the amount of work done. Even to make a rough approximation, it would be necessary to look closely into the detail of every design. It seems, therefore, impractical to attempt to get a solution by the theorem of least work, such a favourite resort when we are in difficulty and cannot see a ready solution by other means. Seeing that no accurate solution can be obtained on those well-known lines, though fully aware of the deficiencies of the ordinary methods the author has demonstrated and used in practice the ordinary customary methods of calculation, as given in text-books, reports, regulations and the like. It is only within the last few years that he has been able to arrive at a simple mode of calculation which is considered to be sufficiently accurate for all practical purposes, and to

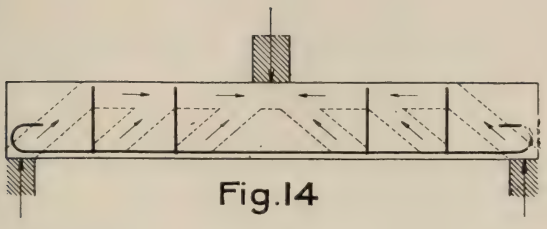


Fig.14

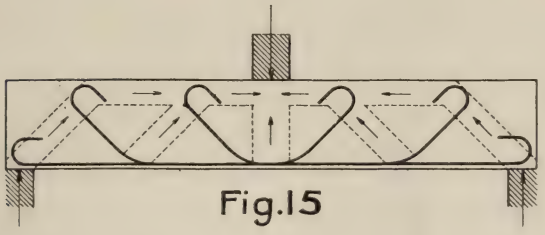


Fig.15

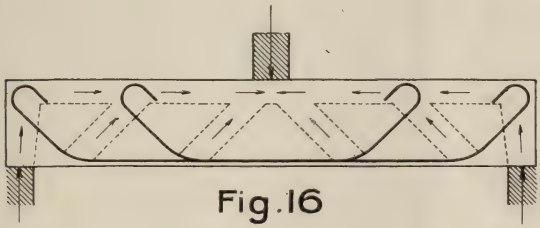


Fig.16

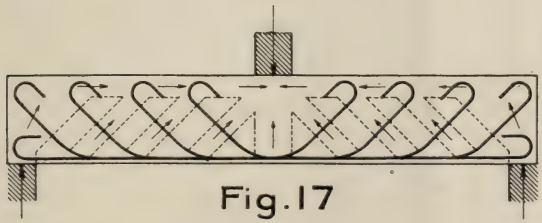


Fig.17

be free from the mathematical inconsistencies of the current theory.

In reinforced concrete calculations, as customarily performed, we are familiar with the fact that a formula for the strength of reinforced concrete beams has been worked out on various assumptions in which a co-efficient is inserted to express certain relations, this being the so-called modular ratio. Loading tests on beams have been conducted, and from the records of such experiments a value has been adopted for this co-efficient for the particular quality of concrete employed, and for moderate amounts of steel which fits the formula somewhat to the experimental results of those tests. Surely this is an artifice! It means that the co-efficient in our formula is derived empirically The ordinary formula is called rational, as distinct from empirical, but if the reasoning is not logical, and the assumptions do not fit the facts then it is hardly rational, and seeing that dependence is placed upon a co-efficient derived from experiments, it would be permissible to refer to the whole method as empirical.

Reverting to our illustration of the cantilever and beam referred to in respect to Figs. 6 and 7, the same simple mechanics apply to the case of the reinforced concrete arch or frame. The resistance moment is equal to the bending moment, and the total compression must equal the total tension. If the concrete is cracked and therefore unable to resist tension, the tension must act at the centre of the steel reinforcement. The centre of pressure is not so obvious nor so easily obtained. The problem is to determine its position in order to get the arm of the resistance couple.

The same fundamental relations exist in the frame or arch. Fig. 18 shows this. The inclined pressure of the strut is resisted by the horizontal pull in the steel and the vertical reaction of the support. Now, we can replace any force by two or more acting at the same point. At the centre we replace the inclined force by horizontal and vertical forces called the components of the original force, as indicated by the dotted arrows. The horizontal components are the compression, the vertical components are the shear. If, instead of a frame we were dealing with an arch without a tie-rod, the abutment would have to supply the horizontal force equivalent to the tension in the tie-rod as indicated by the arrows at the ends. The tension and compression form the resistance couple, the arm of which is a, as indicated in the diagram.

The distribution of the stress in a homogeneous beam has already been described, and it has been pointed out how

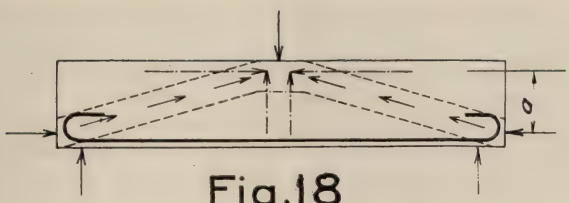


Fig.18

Fig.19 Fig.20 Fig.21

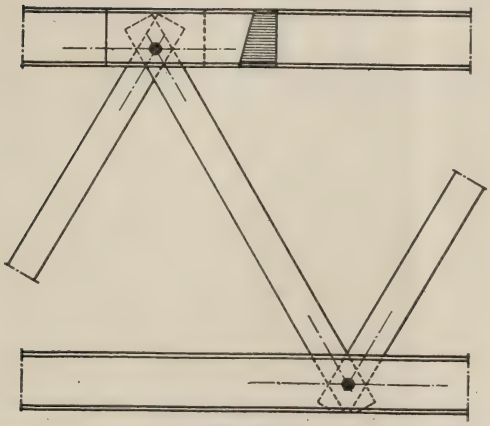


Fig. 22.

it may start by being triangular, as indicated in figure 19, but will change subsequently to something in the form of a semi-parabola, and then a quarter ellipse, like Fig. 20, and may even approach a rectangle as in Fig. 21. What, now, is the probable distribution of stresses in the compressed area of a reinforced concrete member subjected to bending? By analogy with the arch or the frame the stress must be much more uniform from one fibre to another than if we look upon it as a beam of elastic material. In a truss or frame the neutral axis has nothing to do with the distribution of stress. For example, let us consider just a panel of a Warren truss, in which the joints are hinged so that the pressure passes through the hinge. If in the compression boom of such a truss the hinge be placed below the centroid of the section so as to produce an eccentric thrust the stress will be actually greater on the lower edge of the compression boom than it is on the top, as indicated in Fig. 22. In reinforced concrete it has been argued that because cracks result from bending, and run upwards into the beam, just above the point to which they extend the concrete will start to be in compression, and owing to the curvature the outer fibres will be compressed more than those near the middle. This may appear a good argument at first sight, and conflicts with what has just been indicated as to what could happen by frame action. The inclined compression struts and diagonal tension members of a reinforced concrete truss (which is really what a cracked reinforced concrete member becomes), the horizontal components of which create the compression in the boom, certainly appear to communicate their inclined forces to the under-side of the compression boom, so that the first argument would still appear to be somewhat relevant.

Let us see if we have any experimental data to settle these differences. The majority of bending tests on reinforced concrete members are on sections reinforced with a small percentage of steel, namely, 1 per cent. or less. With such a limit the reinforcement is the weaker element, and the compression could be safely carried on a small top boom of concrete. Any number of experiments show that, whenever small percentages of steel are used with a fair quality concrete the so-called neutral axis travels upwards, indicating that a smaller and smaller area comes into play to resist the compression. Naturally if the reinforcement is weak it will be stressed beyond the yieldpoint, and will begin to deform plastically, with the result that the beam will go on deflecting, the radius of the curvature will increase, and

cracks open wider, making the area left to take compression less. When rupture finally occurs in such cases we do not, as a rule, find the steel snapped apart, nor, indeed, on examination of the steel out of broken beams do we notice any stricture or waist. The bars, indeed, when tested, appear to be as good as before. We find, therefore, on ultimate rupture, that it is the concrete which appears to be crushed on the concave side whereas the steel was really the primary cause of failure.

By the ordinary theory if the steel becomes overstrained its apparent modulus will be lowered In a beam weak in tension this means that the modular ratio will tend to be reduced. A lower modular ratio means that the neutral axis will rise. So far the ordinary theory agrees, but while the steel is stretching the area in compression is being reduced, and the concrete must be having its modulus reduced also. The modular ratio is therefore an unknown quantity. But so far as small percentages are concerned there is no need to import such a factor into the equations for determining the ultimate strength. We have a sufficient criterion in the fact that the total compression must always equal the total tension. It looks as though Nature enabled an adjustment to take place so as to assist the over-strained steel as much as possible, throwing on the stronger element of concrete a bigger share until finally it, as the less ductile material, crushes. The steel is really the primary cause of failure owing to its weakness, but when ultimate failure does occur, both the concrete and steel are taking the maximum they are capable of developing.

If we know the maximum stresses to be attained in the steel and the concrete and adopt some distribution for the compressive stresses in the fibres we can express all the fundamental relations without recourse to the modular ratio. Thus if we say the distribution is triangular then for a rectangular section $C = \frac{1}{2}bcn = \frac{1}{2}\ bcdn_1$, while $T = tA = tA_1 bd$ But $C = T$, therefore $n_1 = 2A_1 \frac{t}{c}$ where t and c are maximum stresses for the materials.

Now, on the other hand, what happens when the steel is the stronger element? Few tests have been carried out with high percentages of reinforcement, but those that have afford some very striking results. These tests appear to have been overlooked, yet they are of extreme importance. Firstly we would refer to Professor Talbot's experiments of 1904 at Illinois University. In those tests special precautions were taken against failure by shearing or secondary stresses, and

the sizes adopted were such as would approximate to those under ordinary working conditions. The longitudinal deformations during loading were carefully and accurately measured with a view to determining the position of the neutral axis. With this object two extensometers were placed on one side of the beam, one having its contact points $1\frac{1}{4}$ inches below the upper surface and $1\frac{1}{4}$ inches above the lower surface, and the other having its upper contact points at the same level, but the lower points $3\frac{3}{4}$ inches below the lower surface. By this arrangement two values for the elongation of the reinforcements and compression of the concrete were determined, and also the resulting positions of the neutral axis The concrete was mixed moderately wet in the proportion of 1 cement, 3 sand and 6 coarse material. The loads were applied at two points, one-third the length of the span from either support. Reference will be made to two beams No. 27 and 28. Their size was 12 ins. by $13\frac{1}{2}$ ins., the depth to the centre of the reinforcement being 12 ins. The span of the beam was in each case 14 ft. The length of the central portion of the span where the metal had its full section, no bars being bent up, was 60 ins. The age was 63 and 61 days respectively. The reinforcement of beam No. 27 consisted of four $\frac{3}{4}$ in. plain square bars giving a percentage of metal of 1.56. In beam No. 28 six $\frac{3}{4}$-in Johnson Corrugated bars were used giving a percentage of reinforcement of 1.52. The elastic limit of the square bars were stated to be 33,000 to 35,000 pds. per square inch whereas for the Johnson bars the elastic limit was 55,000 to 60,000 pds. per square inch. The ultimate resistance of these two kinds of steel were not stated, but for the mild steel was probably about 50,000 pds./in^2, judging by what Professor Talbot gave for similar steel in the following year's tests. The ultimate strength of the Johnson bars was probably quite 80,000 pds/in .

Fig. 23 shows the movement of the neutral axis in beam No. 27 with mild steel reinforcement, and fig. 24 the movement of the neutral axis in the case of beam No. 28 having the stronger steel. Beam No. 27 carried a maximum load of 26,900 pds. and Beam No. 28 a maximum load of 34,300 pds. We note that in the first case the neutral axis travels upwards at first and then remains fairly even to the end, though slightly dropping, whereas in the second case after travelling slightly upwards it distinctly travels downwards towards the end of the test. What explanation is there for these tests on the ordinary theory? Both steels must have had about the same modulus. The concrete was probably

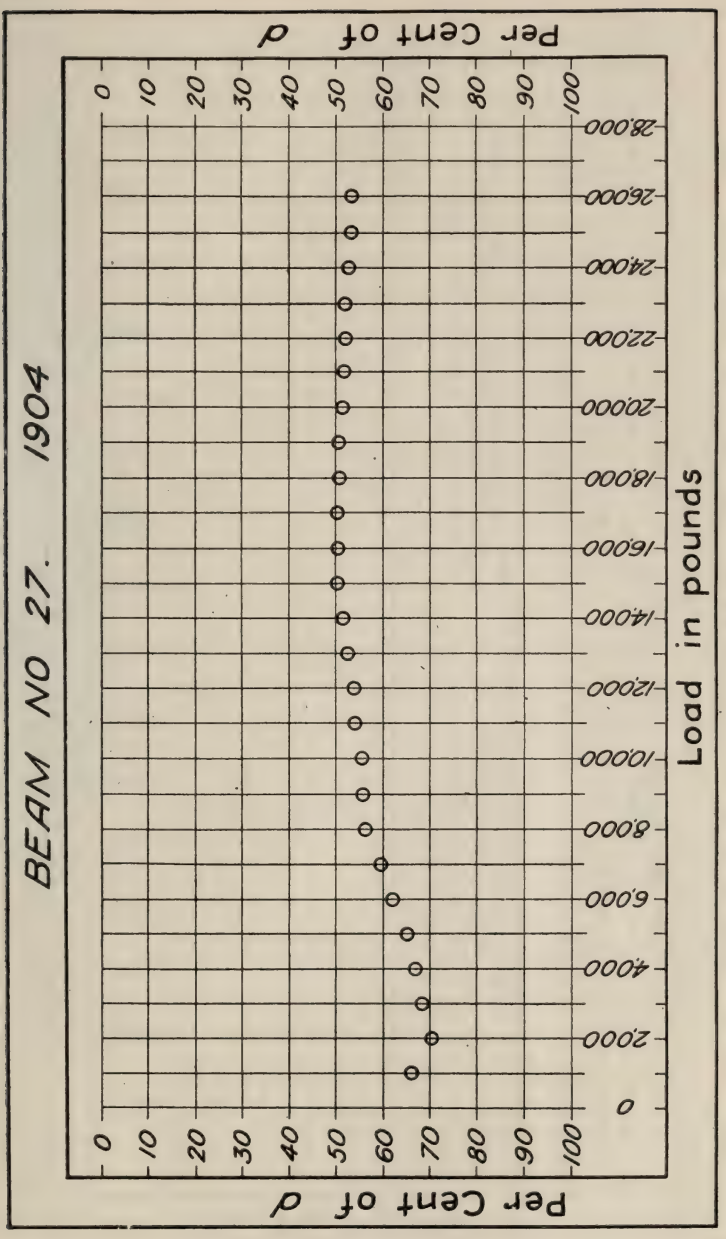

BEAM NO 27. 1904
Per Cent of d
0
10
20
30
40
50
60
70
80
90
100
Load in pounds
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
20,000
22,000
24,000
26,000
28,000

Fig. 25.

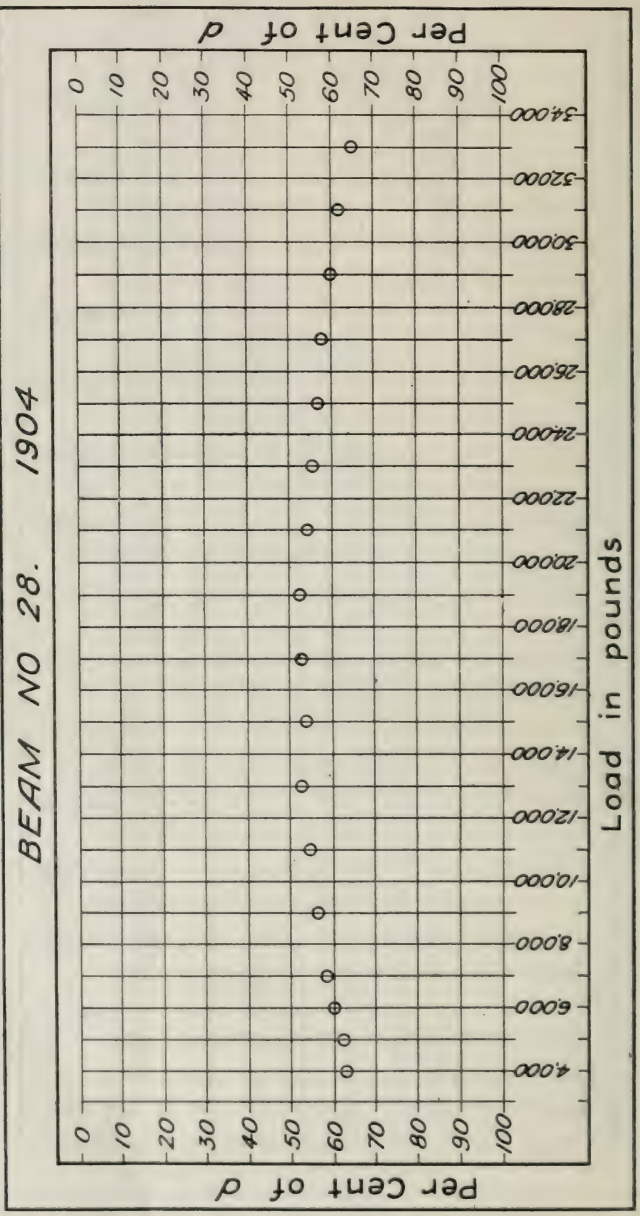

BEAM NO 28. 1904
Per Cent of d
0
10
20
30
40
50
60
70
80
90
100
Load in pounds
4,000
6,000
8,000
10,000
12000
14,000
16,000
18,000
20,000
22000
24,000
26,000
28000
30,000
32000
34,000

Fig. 24.

very similar, and there were practically equal percentages of steel. Yet the positions of the neutral axis differ materially. If in the former case the mild steel is stressed beyond the yield point nearing the breaking load, we should expect, for reasons explained above, to find the neutral axis going up. Instead of that it descends slightly. That looks very much as though the concrete were the weaker element. In the latter test the steel was still stronger, than in the former, and the recorded values show a gradual lowering of the neutral axis. How far the neutral axis dropped at actual rupture could not be observed, of course. The only way the ordinary theory could account for this is on the basis that through overstrain the modulus of the concrete was lowered and the modular ratio increased. But we are usually advised to adopt a set value throughout. Seeing that both steels had the same modulus, and that the concrete was the same in both cases, we would conclude that the ordinary theory would make the modular ratio the same—yet the two results are different, and the only way to attempt to reconcile the ordinary theory with the facts is to adopt a different ratio for each case These two tests bring out that the stronger steel can develop a greater moment of resistance by bringing more of the concrete in to resist compression. The concrete will begin to squeeze, and more and more will come into play; as the outer fibres give way a more equal stress is put on the inner fibres, just like we have seen for homogeneous beams. If, once more, we want to calculate the ultimate resistance moment we need no modular ratio—we can proceed as in the case of small percentages, but there is a limit in that we cannot bring more concrete into play than the total depth of the beam, and if the steel is strong enough to do that we can no longer use the foregoing equation for the neutral axis ratio, nor do we want to do so because n_l becomes simply 1. We but need to know the limiting percentage of steel for any particular strengths of steel and concrete. This will be given by putting $n_l = 1$ in the foregoing formula, and inserting values for t and c; thus for triangular stress distribution

$$n_l = 1 = 2A_l \frac{t}{c} \text{ and } A_l = \frac{c}{2t}.$$

In Professor Talbot's further experiments in 1905 a similarly proportioned concrete was used. Four beams are signalled out for comment here, namely, Nos. 24, 28, 29 and 46. The ages respectively were 63, 60, 62 and 61 days. The beams measured 8 ins. in width, 11 ins. in depth and 13 ft. long, with a test span of 12 ft. The steel reinforce-

ment was placed 10 ins. below the top surface, and consisted of mild steel of two qualities, the bars being mixed indiscriminately. One quality had a yield point of 33,200 pds/in.^{2}, and ultimate of 51,900 pds./in.^{2}, while the other had a yield point of 43,600 pds./in.^{2}, and ultimate of 62,350 pds./in.^{2}. The loads were progressively applied and released in the first two cases, but in the third a load of 12,000 pds. was repeated 15 times. In the fourth case the loading was continuous. In the first two cases the failure was stated to be by diagonal cracks followed by compression. In the third case Professor Talbot states that the final failure of the beam occurred by compression at a time when failure by diagonal tension seemed imminent. In the fourth case failure was by compression in the concrete.

The first three beams were reinforced each with 4-$\frac{3}{4}$ in. plain round mild steel rods, thus giving a percentage of reinforcement of 2.21, while Beam No. 46 contained 5-$\frac{3}{4}$ in. plain round mild steel rods, thus giving a percentage of 2.76.

Figs. 25 to 27 give the records showing the position of the neutral axis at the various loads.

The maximum loads carried were: Beam No. 24, 15,600 pds.; Beam No. 28, 14,300 pds.; Beam No. 29, 15,900 pds.; and Beam No. 46, 15,200 pds. In all cases the neutral axis travelled downwards, and in Beams 29 and 46 reached so near the bottom as 72 and 80 per cent. of the depth. The final position at rupture could not be observed. In these tests we have mild steel in sufficient quantity never to be stressed to its ultimate resistance.

Both in the 1904 and 1905 tests there is a paucity of information about the real strength of the concrete. In 1904 the concrete cubes were stated to be wetted for three days, whereas the curing of the concrete in the 1905 tests were admittedly insufficient. Six cubes in 1904 gave an average crushing strength of 2,030 pounds per square inch. Analysis of the tests would suggest that the strength of the concrete actually in the beams was very low, somewhere in the neighbourhood of 1,300 pds. per square inch. In 1905 the cubes that were crushed did not relate to the particular beams that are here signalled out; the cubes that were tested, however, relating to other beams of the series, gave such figures as 777, 816, 1,105, 1,310, 1,355, and 1,590 pds. per square inch. Not only were no proper precautions taken to prevent the beams and cubes drying out too quickly, but the beams were handled and stacked at an early stage, and two of the stacks collapsed breaking many of the beams, and the cracked ones as well were tested. In the beams

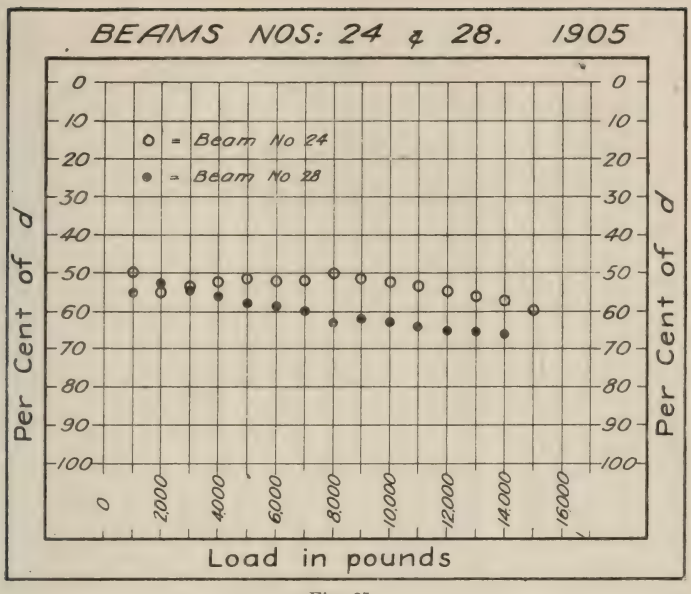

BEAMS NOS: 24 & 28. 1905
O = Beam No 24
● = Beam No 28
Per Cent of d
Load in pounds
0
10
20
30
40
50
60
70
80
90
100
2000
4000
6000
8000
10000
12000
14000
16000

Fig. 25.

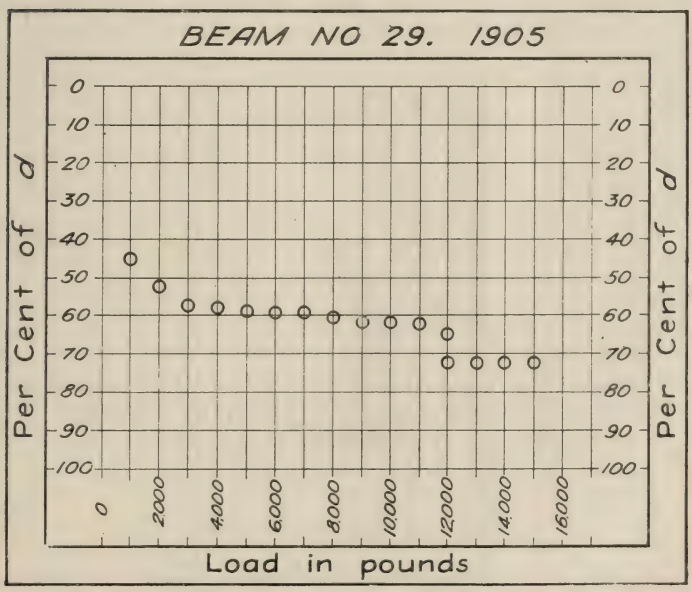

BEAM NO 29. 1905
Per Cent of d
Load in pounds
0
10
20
30
40
50
60
70
80
90
100
2000
4000
6000
8000
10000
12000
14000
16000

Fig. 26.

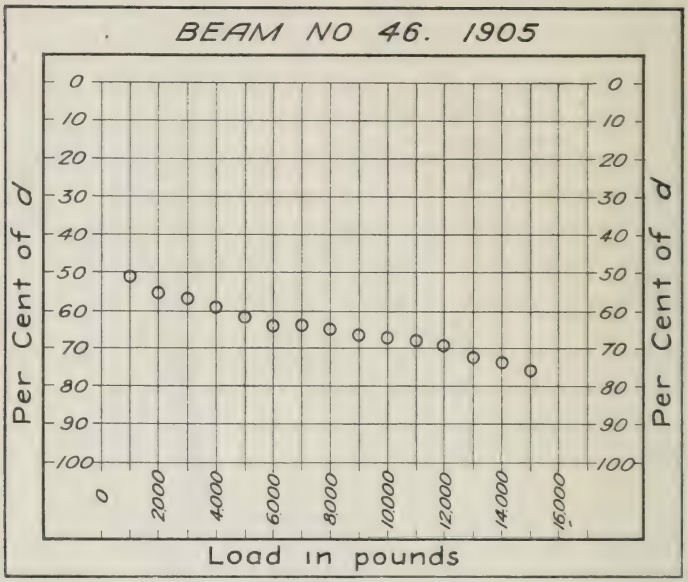

BEAM NO 46. 1905
Per Cent of d
0
10
20
30
40
50
60
70
80
90
100
0
2000
4000
6000
8000
10000
12000
14000
16000
Load in pounds

Fig. 27.

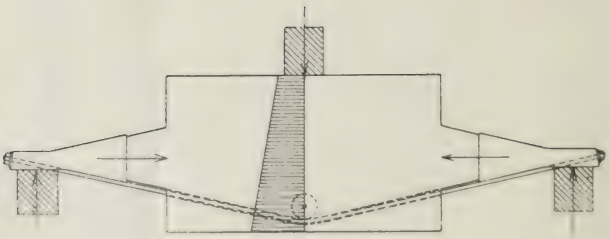


Fig. 28.

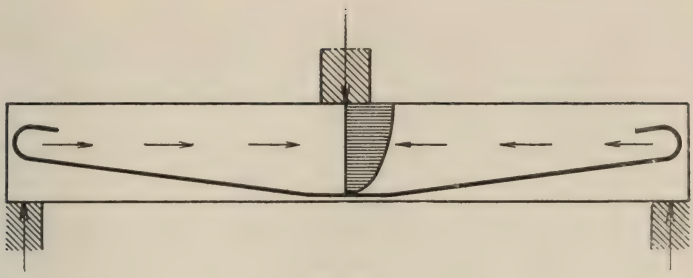


Fig. 29.

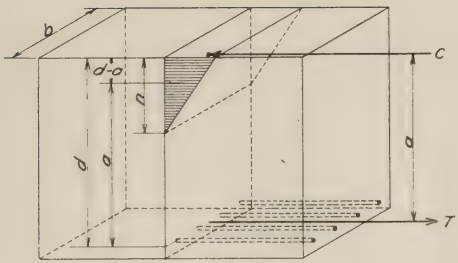


Fig. 30.

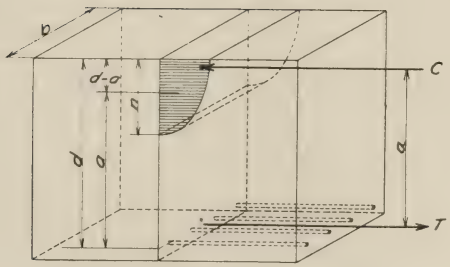


Fig. 31.

of this second series the final position of the neutral axis at rupture as in the former set is not directly ascertained, but in Beams Nos. 29 and 46, the position of the neutral axis is recorded as being near the bottom of the beam, and at rupture it would be still closer.

The test of Beam No. 29 is interesting because of the fact that a load of 12,000 pds. was repeated 15 times. The effect of this repeated loading is clearly shown on the diagram by the upper and lower readings for that loading. This test brings out the point that the successive plastic deformations produced by re-loading in this way merely made the member settle down to its work, putting it properly into the condition of a simple frame or truss, in which the distribution of stress in the compression boom would be more uniform than it was in the early stages of loading. That in itself puts a reinforced concrete "beam" in a different category from a homogeneous beam made of material whose tensile is different from its compressive strength, of which it has been explained that the plastic deformation due to re-loading might result in eventual rupture in tension, whereas in the reinforced concrete beam the repeated loading has no deleterious effect, merely serving to make the stress more uniform even than the assumed elliptical stress distribution, tending to give a rectangular stress distribution to which frame or truss action directly points.

In the first report sheets Professor Talbot regarded Beam No. 29 as failing by diagonal tension, but as stated above he expresses his final opinion that the failure was through compression. Seeing that Beam No. 24 carried 15,600 pds., which is near that carried by Beam No. 29, it is probable that the failure in that case also was by compression, and this inference receives confirmation from the fact that he reports the failure of Beam No. 46 as caused by compression, although it carried a still lower load, namely, 15,200 pds. Beam No. 28 does not fall into line in this respect. The concrete must have been poorer than in the case of the other beams because the arrangement of reinforcement was precisely similar to Beams Nos. 24 and 29. It is possible that with such poor concrete the beam failed by shear, and consequently the neutral axis was not observed to drop down so low as in the other examples.

If cracks had been developed near the middle of the beams Professor Talbot's tests would indicate they would be closed up, as more and more concrete was thrown into play. So that indeed the whole section could, if the design were such as to warrant

it, come into play. This rather startling proposition may be made clearer by giving some diagrams to show what kind of construction could enable the whole section to be stressed. If we imagine a simple frame or truss like fig 28, in which are provided special shoes at the end, with a wheel to allow the steel to slip readily at the middle, it becomes clear that the thrust in the compression boom could be applied eccentrically, so as to make the stress at the bottom actually greater than at the top, as indicated in the diagram. Fig. 29 shows quite a practical scheme for an ordinary reinforced concrete beam in which the stress will obviously be something in the nature of that again indicated by hatching.

It was explained before that the very simple mechanics of Figures 6 and 7 apply equally to a beam, arch, frame and truss, so we can ascertain all we want if we know the arm depth. In a reinforced concrete member the maximum pull will come through the steel bars, and we know the centre of tension, therefore. Now, as regards the compression. If we choose any particular distribution of stress across the section we can easily compute the centre of gravity, i.e., the centre of pressure in terms of the depth of the member subjected to compression. If we have regard to ultimate loading we should be justified in adopting a stress figure like the quarter ellipse, for the case of ordinary homogeneous beams. It has already been mentioned, however, that under reloading, this might not be always true of homogeneous beams, though correct for the first time of loading, and possibly for a few successive loads. In a reinforced concrete member we have seen that the stresses may vary as a trapezium shaped figure or rectangle. The ordinary theory would ask for a triangle. The author favours an ellipse because that is probably a close enough approximation to the true distribution, yet a safer assumption to make than to assume a trapezium or a rectangle. It should be borne in mind, however, that under special conditions such a stress figure would be warrantable, and might be adopted with proper safety. The elliptical stress distribution is favoured because as a simple figure it approaches a rectangle or trapezium on the one hand as seems most appropriate to truss action, and on the other hand falls into line with the probable stress figure for homogeneous beams tested to destruction without repeated loading, and so will serve to make the student familiar with the properties of an elliptical figure for analysing tests to destruction of beams of homogeneous material. At the same time it is a safe and simple assumption to make for the practical design of reinforced

structures. For those, however, who would care to assume a triangular stress distribution or a parabolic stress distribution, similar properties are here set out with diagrams.

If a large proportion of steel be inserted in a reinforced concrete member, we may expect more and more of the concrete to come into play to resist compression, but there must come a point where the whole section will be acting. The author feels inclined at present to limit this to the maximum effective depth, and not to take into account any concrete covering to the steel, although Fig. 28 would suggest that in special circumstances such would be warranted as affording greater total compression. If we adopt this procedure in practical calculation the depth of the compression boom cannot become greater than the effective depth. Up to a certain percentage of steel rupture will take place when both concrete and steel are stressed to their limits, the requisite amount of concrete being brought into play so that the total compression equals the total tension—our old fundamental mechanical principle. When, however, the whole depth of concrete is brought into play down to the steel, no more can be got out of the concrete, the arm is at its shortest and the tension its greatest. This condition will give the maximum resistance moment in compression. If the steel is in greater proportion than merely what is necessary to balance this compression the stress on the steel will be lower than the ultimate. Such considerations indicate clearly that there is a maximum amount of steel that we can usefully insert in a beam. It is desirable to emphasise this point, because the other customary theory would indicate that there was always a certain amount of advantage as regards strength to be derived from putting in a greater amount of steel.

The following data relate to the author's theory:—

Symbols: (Referring in some instances to Figs. 30 and 31)

A = area of tensile reinforcement (in sq. ins.)

A_f = areal ratio, i.e., ratio of A to bd, therefore $A_f = A/bd$ and $A = A_f bd$

a = arm of resistance moment (in ins.)

α = arm depth ratio = a/d $\therefore a = \alpha d$

B = bending moment of the external loads or forces (in pds. ins.)

b = breadth of rectangular section or flange of a tee-section (in ins.)

c = compressive strength (maximum) on the compressed edge (in pds./in.^{2})

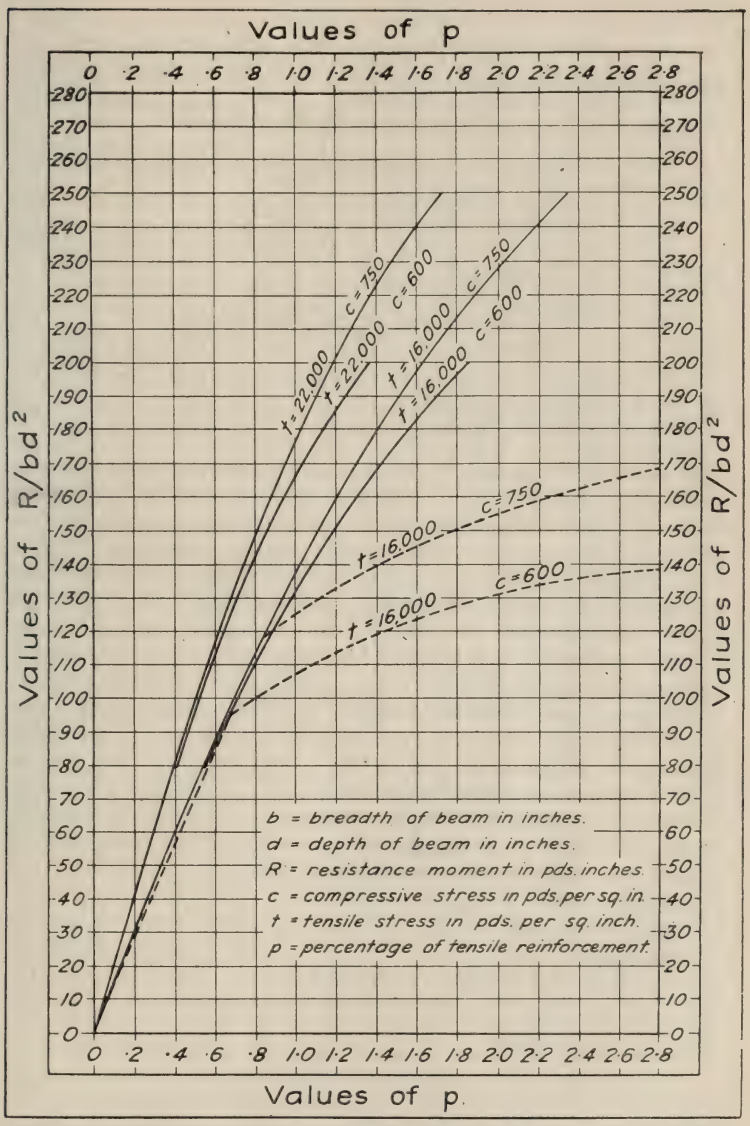

Values of p
0 ·2 ·4 ·6 ·8 1·0 1·2 1·4 1·6 1·8 2·0 2·2 2·4 2·6 2·8
Values of R/bd²
c = 750
c = 600
t = 22,000
t = 16,000
b = breadth of beam in inches.
d = depth of beam in inches.
R = resistance moment in pds. inches.
c = compressive stress in pds. per sq. in.
t = tensile stress in pds. per sq. inch.
p = percentage of tensile reinforcement.

Fig. 32. Comparing theories. Dotted lines show values given by the L.C.C. Regulations, and Full lines values given by the author's method.

C = total compressions. i.e. summation of compressive stresses

d = depth (effective) of member (in ins.), i.e., distance from extreme compressed edge to centre of tensile reinforcement

l = length of the effective span of a structural member subjected to bending (in ins.)

n = distance to neutral axis or to limit of compressed part from extreme compressed edge

n_l = neutral axis ratio = n/d $\therefore$ $n = n_l d$

R = resistance moment of the internal stresses in the bent member (in pds.-ins.)

R_c = resistance moment of compressive stresses

R = resistance moment of tensile stresses

T = total tension, *i.e.*, summation of tensile stresses

t = tensile stress (maximum) on the extended edge or on the inforcement (in pds./in.^{2})

W = total weight to be carried on the member (in pds.)

It will be noted that as the compressive stress and tensile stress up to the limiting percentage of steel will reach their ultimates at the same time, we can with that limitation get the simple formula Number 9 in the above table. From this data Fig. 32 has been plotted for triangular stress distribution for comparison with the ordinary theory. The dotted line shows the resistance moments obtained with certain percentages of steel at the working stresses stipulated in the London County Council Regulations. For the critical percentage, as it has been called (and by some persons the economical percentage), namely, that percentage of reinforcement with which the stress in the steel and the stress in the concrete reach their limits, both theoretical treatments get the same results. Below the critical percentage there is very little difference as regards the moment of resistance, and most structures that have been built of recent years have been designed with the steel limited to the critical percentage or less. It is only when working on big percentages or with specially strong steel that the author s theory departs so greatly from the other. This is shown on the diagram by the curves above the critical percentage of the ordinary theory.

With the ordinary theory the economical limit is below where the formula for use becomes that for the moment of resistance in compression. The steel, according to that theory, above that limit cannot be strained to the full, though the concrete can. There is, therefore, very little advantage gained from increasing the amount of steel beyond the critical

percentage. In the author's theory, however, the increasing resistance moment obtained from an increasing amount of steel goes on much as before, though with some reduced effect, the line curving round gradually. The author's line, however, does not go on interminably like the other It has to be stopped when a percentage is reached that gets the maximum compressive resistance out of the concrete. Thus it shows the point at which if any more steel is inserted it will give no greater strength—a fact with which most reinforced concrete specialists are familiar in practice.

Figs. 33 to 36 were plotted from the formulæ set out above and may be used in design. They are more applicable, however, for analysing the stresses in tests in the manner explained below.

One criticism may be that the author's method is all very well for the computation of the ultimate resistance moment, but it does not afford a means of determining working stresses with loads considerably less than the ultimate, and that the ordinary theory, however far wrong it may be as regards the ultimate strength, does tend to give some indication of the working stresses at working loads. This might be said to be a justification for the adoption of the factor termed the modular ratio, though seeing that with an ordinary working stress of, say, 16,000 lbs./in.^{2} in the steel the concrete must be cracked, and therefore a reinforced concrete member must be acting as a frame or truss rather than a beam, any calculation based on assuming it to be what it is not must be erroneous.

If, however, we are abolishing the use of the modular ratio what are we to put in its place to afford some idea of the working stresses induced by any load considerably less than the ultimate? The author wishes to meet, if he can, the prejudices of those trained to thinking in terms of working stresses rather than in working loads determined by taking factors of safety upon the ultimate strength. Fig. 37 is prepared with that object. From an inspection of tests such as those of Prof. Talbot, it would appear that it is sufficiently accurate to assume that, for either small or large percentages of reinforcement, at very early stages in the loading the arm will approximate to .82 of the effective depth. If the distribution is triangular that means that the compression will be taken on .54 of the effective depth. If the beam were of homogeneous material the amount in compression would be .5 of the depth. The provision of steel reinforcement in itself causes the neutral axis to come down. Then, again, the concrete on the underside of the

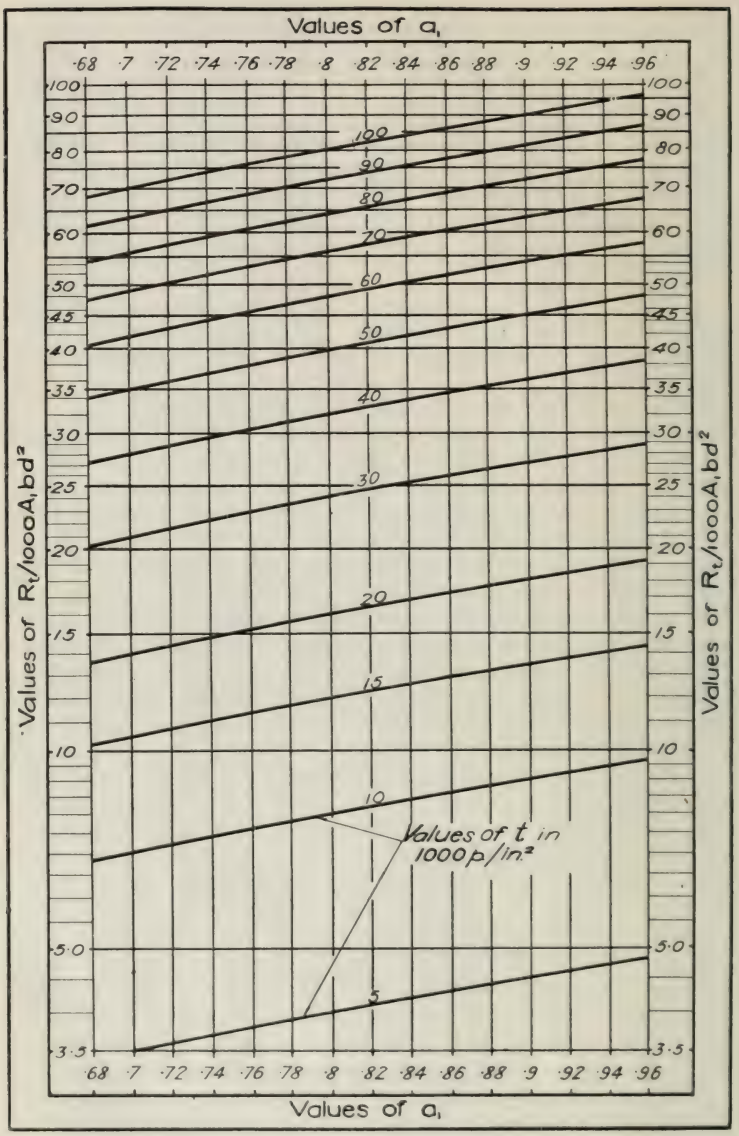


Fig. 33. For tensile resistance moment.

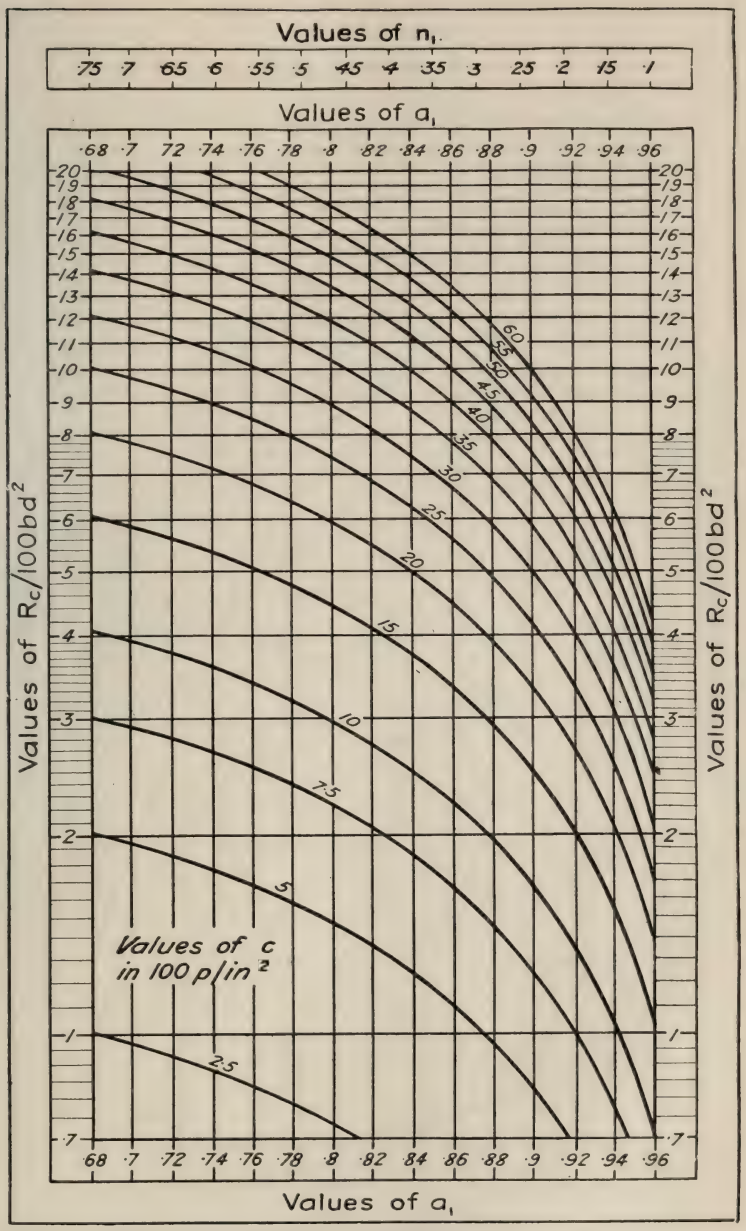


Fig. 34. For uitimate compressive resistance moment (elliptical stress distribution).

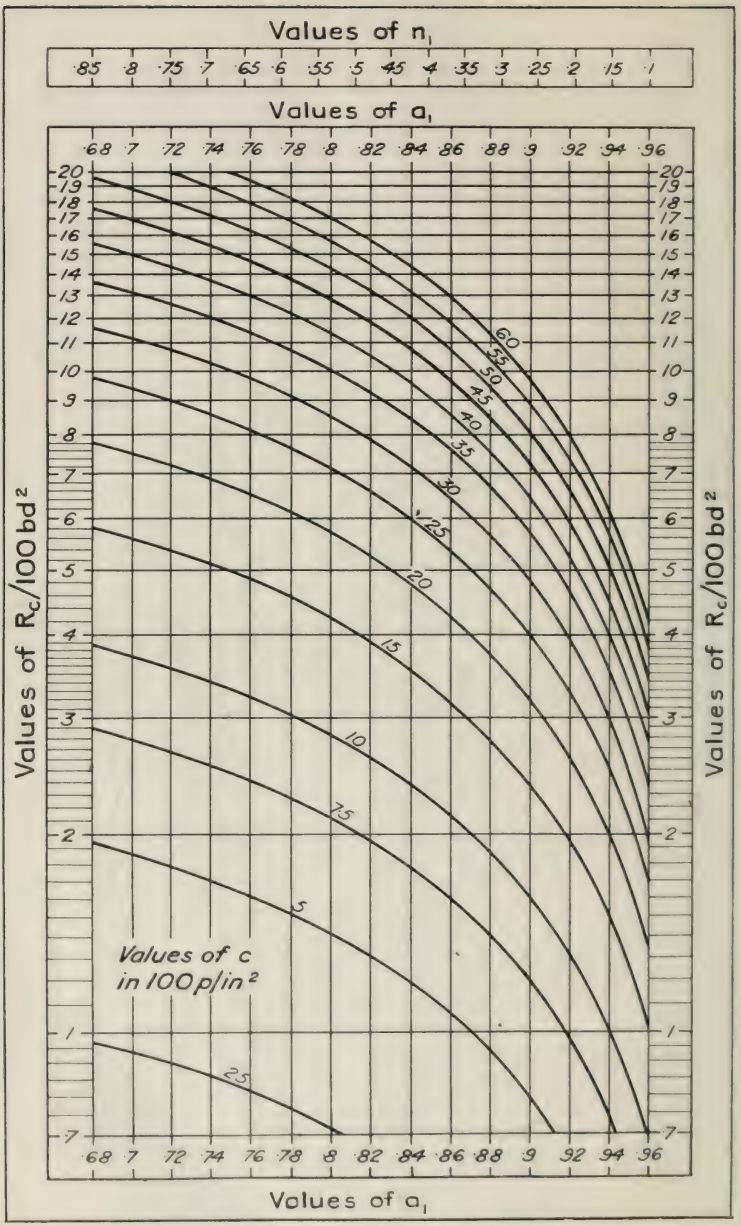


Fig. 35. For ultimate compressive resistance moment.
(Parabolic stress distribution.)

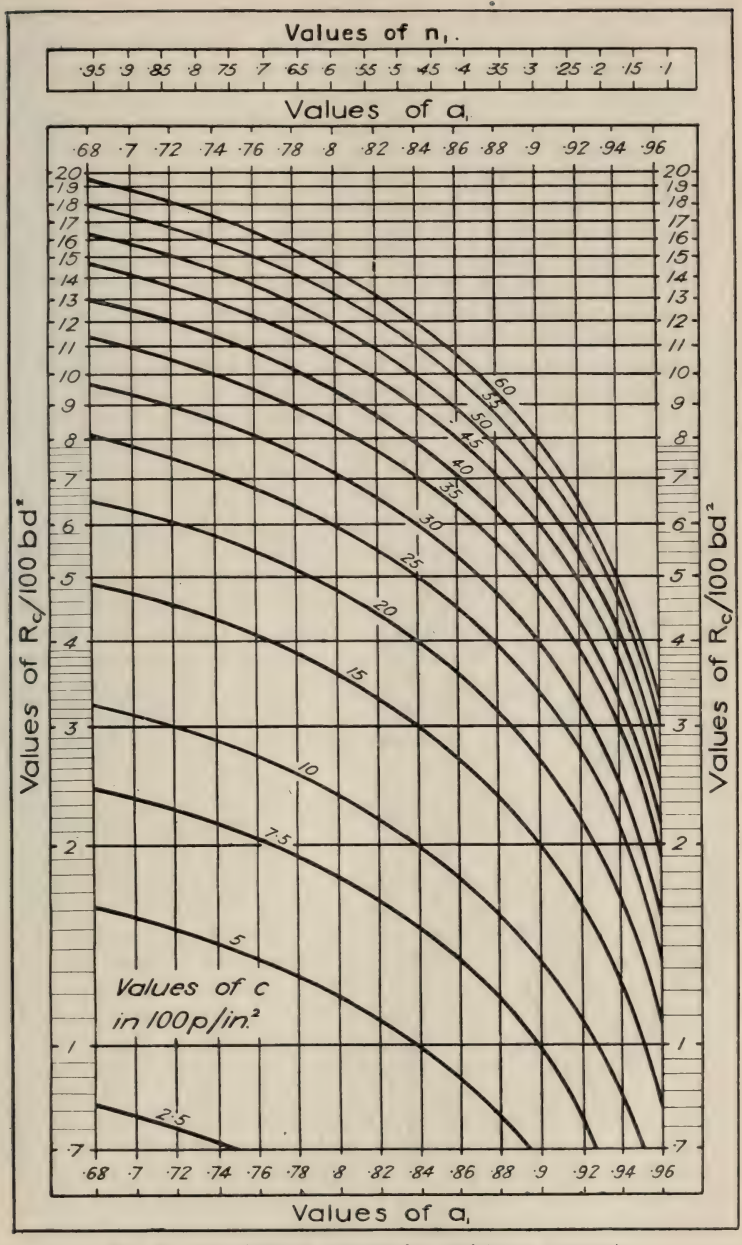


Fig. 36. For ultimate compressive resistance moment (triangular stress distribution).

bars will act also to bring the initial position of the neutral axis lower than .5 of the effective depth. In the early stages of loading the position of the neutral axis will keep near to the initial position. In the final stages of loading, however, the depth of the part in compression will be near that calculated by the author's method. So if we find the ultimate arm by that method and take .82 as the initial arm depth ratio we can pick out the proper line from Fig. 37. From a study of tests the author suggests that the variation of the arm at intermediate stages should be sufficiently nearly given according to the curves shown in the figure which is drawn so that when the working load is .4 of the ultimate, or, to put it another way, when the factor of safety is 2.5, the depth of the part in compression will be half-way between the initial position of the neutral axis and the final position of the compressed depth. Between the terminal points curves are drawn as segments of parabolas, with the apices at the initial and final points. It is suggested that for early stages of loading a sufficiently accurate result in analysing tests will be obtained by using this diagram in connection with Fig. 36, which relates to triangular stress distribution. At a middle stage Fig. 35, relating to parabolic distribution, would be more accurate, while in the final stages of loading Fig. 34 for elliptical stress distribution should be used. For those who wish to design for working stresses, in the usual way, and to adopt a triangular stress distribution, Fig. 32—or other diagrams drawn in a similar way—may be adopted. In all cases, however, Fig. 33 for the Moment of Resistance in tension applies.

Now, apart from what has been said, are there any other special novelties to note with this new theory? The most interesting feature to the author's mind is one conclusion that it leads to, namely, that the modular ratio of the concrete and the steel can be consigned to limbo. All that matters is that the steel shall be capable of carrying the tension and the concrete the compression. Thus, as the steel has merely to carry the total tension, if we were to use very strong steel we could insert quite a small amount, and it would serve just as well as a large quantity of a weaker steel. That is a justification for those who have been urging for many years the advantages of such special steels. Tests have been made with drawn wire fabric, by firms manufacturing such material, which clearly indicated that there was some virtue in such materials, for smaller quantities gave as good results as larger quantities of mild steel, and alternatively if the same quantities were inserted, those

slabs in which drawn steel were employed carried bigger loads than those reinforced with mild steel. Specially strong steels are generally more expensive than ordinary mild steel, but their extra expense appears to be warranted by the economy that they effect.

The Concrete Institute, it will be remembered, appointed a special Committee, of which the author is the Chairman, to investigate this subject, and tests were carried out at the Manchester School of Technology on slabs reinforced with special steels of various kinds, the results of which were recorded in a paper read by the author last session. Although his mind had been running on this subject for a good many years past it was the formation of that Committee and the formulation and carrying out of its experiments which caused him to concentrate attention on the subject, and led him to arrive at this theory here set forth.

Another striking feature is that, as shown in Fig. 32, whereas the ordinary theory indicates that for fairly high percentages of steel the moment of resistance is very little increased beyond that at the critical percentage, the new theory makes the moment of resistance increase almost as rapidly as before. This indicates that we can use with advantage more mild steel than we have been in the habit of using, or, alternatively, we can use, say, the same quantity as we have been using, but stronger steel, and that we can with economy reduce the depth, or alternatively with the same depth, adopt greater spans. The following is an example of slabs computed by the two methods, adopting both triangular and elliptical stress distribution. The cost is given in reference to certain basis prices for steel and concrete; of course, in each case the centering will be the same, so that can be eliminated from the comparison.

Example—

Freely supported slab under evenly distributed load to carry a superimposed load of 150 pds. / ft.^{2} on a span of 12 ft.

Dead load of Finish, say 10 pds. / ft.^{2}

Weight of concrete = 144 pds. / ft.^{3}

Factor of safety required = 4

Limits of ultimate stress c = 2,400 pds. / in.^{2}
t = 64,000 pds. / in.^{2}

Unit cost : Concrete 2s. per ft.^{3} = 1d. per ft.^{2} $\frac{1}{2}''$ thick
Steel, 2d. per pd. = 6·8d. per ft.^{2} for every in.^{2} of steel required per ft. width of slab

	Thickness of Slab (in inches).						
	8	7½	7	6½	6	5½	5
Bending Moment =	55,300	54,000	52,600	51,400	50,000	48,800	47,500
d =	7·25"	6·75"	6·25"	5·75"	5·25"	4·75"	4·25"
R/bd^2 =	87·7	98·5	112·5	129·5	151	180	219
A, for ordinary theory ...	·61	·73	1·17	1·87	4·0	12·5	Beyond limit
Cost for Triangular distribution ...	16+3·6=19·6	15+4·03=19·03	14+6=20	13+8·8=21 8	12+17·1=29·1	11+48·5=59·5	
A_1 for Author's theory for triangular stress distribution ..	·6	·7	·82	·98	1·2	1·57	Beyond limit
Cost for ditto...	16+3·56=19·56	15+3·96=18·96	14+4·2=18·2	13+4·6=17·6	12+5·1=17·1	11+6·1=17·1	
A for Author's theory for elliptical stress distribution..	·6	·683	·79	·93	1·12	1·4	1·85
Cost for ditto...	16+3·56=19·56	15+3·76=18·76	14+4 04=18 04	13+4 4=17 4	12+4·8=16·8	11+5·4=16·4	10+6·4=16 4

The figures for elliptical stress distribution are inserted merely for comparison. The author would, in practice, reduce the permissible stress on the concrete because tests on cylinders or prisms, whose height is at least twice the width, show that the true crushing strength is about 80 per cent. of the values derived from cubes. The effect of working the lower stress in this case is to give the same values as for triangular stress distribution.

It will be seen from this that there is a considerable economy effected by reducing the depth and increasing the quantity of steel. That happens to be what was advocated by early pioneers. In early designs the slabs were made much thinner than we have been doing. Cottançin, who built some wonderful structures in Paris and elsewhere, used thin slabs about 2 inches thick for quite considerable spans, and indeed was singular in claiming that reinforced concrete was one of the lightest forms of building construction known, whereas we have been accustomed to consider it as rather heavy. This newer theory would find some explanation for Cottançin's work and his contentions.

In a rectangular beam if the concrete is of at all decent quality, 2 per cent. will not be sufficient to get the maximum out of the concrete. Herr Fritz von Emperger, the Editor of "Beton und Eisen," has been advocating for many years past the use of small experimental beams made by the workmen on actual jobs and loaded by means of a platform suspended by chains passing over the beam, his contention being that the superintendent and workmen by observation of what they can get out of the concrete they are making, soon learn to improve the quality of the work. He found by trial that to make sure of causing failure in the concrete and obviating any question of the steel being overstrained, it was necessary to employ about 4 per cent. of steel. Fig. 32 explains why that is so.

(*To be continued.*)

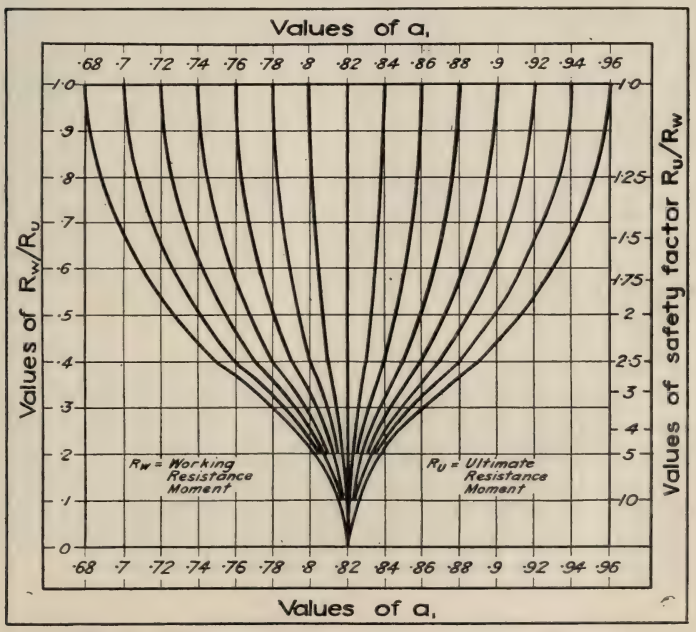

Values of a,
·68 ·7 ·72 ·74 ·76 ·78 ·8 ·82 ·84 ·86 ·88 ·9 ·92 ·94 ·96
1·0 ·9 ·8 ·7 ·6 ·5 ·4 ·3 ·2 ·1 0
Values of R_w/R_u
1·0 1·25 1·5 1·75 2 2·5 3 4 5 10
Values of safety factor R_u/R_w
R_w = Working Resistance Moment
R_u = Ultimate Resistance Moment
Values of a,

Fig. 37. For determining arm with varying safety factors (*i.e.*, ratios of ultimate to working resistance moments).

NEW BUILDINGS IN WHICH CONCRETE, ETC., IS BEING USED.*

PECKHAM. L.C.C. Sumner-Road Elementary Schools. Reinforced concrete floors. Architects: L.C.C. Builders: J. & C. Bowyer, Ltd., Weston Street, Upper Norwood, S.E.

BATTERSEA. Extension to the drapery stores of Messrs. David Thomas, Ltd., 55, Falcon Road. Three-storey building of steel and reinforced concrete. Builders: Bainbridge & Son, Eastbourne; Constructional Engineers: Young & Co., 6, Queen Anne's Gate.

CAMDEN TOWN. Branch for Messrs. English Jersey Breeders Dairies, Ltd. Reinforced concrete floors. Architect: Needham Wilson, 9, Old Square, Lincoln's Inn, W.C. Builders: Bovis, Ltd., Upper Berkeley Street, W.1.

EAST CENTRAL. Block of offices for Messrs. Babcock & Wilcox, Ltd. Reinforced concrete floors. Architect: Victor Wilkins, 12, York Buildings, Adelphi, W.C. Builders: John Greenwood, Ltd., 12 and 14, Arthur Street, London Bridge, E.C. 4.

WEST CENTRAL. Shops and Showrooms in Great Portland Street. Reinforced concrete floors. Architects: Trehearne & Norman, Windsor House, Kingsway, W.C. 2.

PUTNEY. Petrol-filling Station in High Street, Reinforced concrete roof. Builders: George Greenwood & Sons, 301, Brixton Road, S.W. 9.

FULHAM. Swan Brewery in Fulham Road. New copper house of reinforced concrete and concrete blocks, for the City of London Brewery Co., Ltd. Architect: Gordon Stanham, 26—27, Bush Land, Cannon Street, E.C. 4. Builders: George Parker & Sons, Ltd., 124, Sumner Road, Peckham, S.E. 15.

---

* Members desirous of visiting any of these works are advised to make application to the Architect or to the Builders where the Architect's name is not mentioned.

Information regarding the use of reinforced concrete in new buildings, together with the names and addresses of the Architects and Builders, will be welcomed, and should be sent addressed to the Secretary of the Concrete Institute.

PROCEEDINGS.

The 110th Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W. 1, on Thursday, 23rd March, 1922, at 7.30 p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous meeting as published in the *Journal* were taken as read and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association :—

Members.

BARNES, ALFRED HENRY, Lic.R.I.B.A., 14, Sydenham Road, Croydon ; HODGE, Colonel ARTHUR, D.S.O., M.C., Southampton House, 317, High Holborn, W.C. 1. ; PINDER, FREDERICK NAPIER, M.S.A., 214, Manor House Lane, Holme Stack, Preston, Lancs.

Associate-Members.

BISWAS, JOTINDRA NATH, M.R.San.I., Boro' Engineers' Office, Town Hall, Croydon ; COUNSELL, ROLAND OLIVER, c/o Federated Engineering Co., Kuala Lampur, F.M.S. ; FINER, HENRY JAMES, Newhall Chambers, Newhall Street, Birmingham ; MITCHELL, ARTHUR JAMES, A.M.Inst.C.E., 1, Rue Ancienne Bourse, Alexandria, Egypt.

Resignation.

The Secretary announced the acceptance by the Council of the resignation of W. SEWALL, Member.

A paper was then read by Mr. S. F. STAPLES, M.Inst.C.E. M.I.N.A., on "FLOATING DOCKS." At the conclusion of the discussion which followed, a hearty vote of thanks was accorded to the lecturer.

The proceedings then terminated.

The Editor, *The Concrete Institute Journal.*

Dear Sir,

In the March, 1922, *Journal* of the Institute, wherein we have the very excellent paper by E. B. Moulin, M.A., on Capillary Canals in Concrete, there is, I believe, one point which has been overlooked when dealing with the large number of spherical cavities found when a cement block is cut through and its surface ground to a dead flat. These cavities are stated to be probably due to air bubbles carried into the mass while mixing, or, according to the Author, by excess water in the form of drops. I think it highly probable that both theories are, to a certain extent, correct, but the main cause of their formation is the air absorbed by water at atmospheric pressure. When the mass sets this air is left behind in innumerable small air bubbles in exactly similar fashion to the formation found in a block of ice when this is made of undistilled water, the ice being quite white in appearance from the small air cavities formed throughout its mass. To form crystal or clear ice only distilled water is used, or in special cases the freezing takes place from the bottom of the block and the water is gently agitated to free it of air as the mass solidifies. Neither of these methods are applicable to concrete work ; at the same time it is obvious that the smaller the quantity of water used in mixing the fewer spherical cavities will be formed and the more dense will be the mixture.

Yours faithfully,

T. D. Key.

C/o Alexandria Water Company,
Alexandria, Egypt,
24th April, 1922.

CONCRETE INSTITUTE MASONIC LODGE.

Some members of the Institute decided, at a meeting held on March 9, 1922, to present a petition to Grand Lodge for the founding of a Concrete Institute Lodge. Members interested should communicate with Captain M. G. Kiddy, at 296, Vauxhall Bridge Road, S.W.1.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *JUNE, 1922.* **No. 6.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

Papers for Session 1922-23.

Members desirous of reading papers before the Institute during the Session 1922-1923 are invited to communicate with the Secretary of the Institute, at 296, Vauxhall Bridge Road, S.W.1. The title of the paper should be stated, a summary of the subject matter should be enclosed, and a suitable date should be intimated.

Papers should not exceed 10,000 words in length for printing, and are subject to approval by the Literature Standing Committee.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s. 0d.
		Remainder ...	£2	2s. 0d.
ASSOCIATES	,, ...		£2	2s. 0d.
ASSOCIATE-MEMBERS	,, ...		£1	11s. 6d.
GRADUATES, STUDENTS	,, ...			10s. 6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

WHAT IS THE USE OF THE MODULAR RATIO?

PART III. *(Conclusion.)*

By H. KEMPTON DYSON.

TEE BEAMS.

In the current theory a great deal is made of elaborate formulæ for the resistance moments of T-beams. It would be easy for the author to provide formulæ of a similar kind devised in conformity with his particular theory, but from a practical point of view such formulæ are not of general application and can be omitted. As regards T-beams, it is common knowledge to designers that it is economical as a rule to make T-beams as deep as head room will allow. The result is that the concrete of the flange of a T beam is sufficient to take all the compression at a stress which is often a good deal less than the maximum allowable. The percentage of steel employed is therefore comparatively small, such percentage being reckoned in regard to a nominal area found by multiplying the breadth of the flange by the effective depth of the beam. Therefore, in an ordinary economical design a low percentage of steel will be employed, giving as a rule a low stress even according to the ordinary theory; by the author's method the flange will always be ample to take compression, and the part in compression will be but a small portion of the depth of the slab, that is to say, the extent of the compression will be less than the depth of the slab, so that we are able to calculate as though a T-beam were a simple rectangular beam, and the foregoing diagrams for rectangular sections apply to tee beams in practice. So far the formulæ are simple, and at the same time it is contended they are more accurate than the usual.

DOUBLY REINFORCED BEAMS.

Now, what is to be said as regards sections reinforced in compression? Space forbids much being said on doubly re-

No.	Function.	Form of Stress Distribution		
		Elliptical.	Parabolic.	Triangular.
1	C	$\frac{\pi}{4}\, bcn = \cdot 7854 bcn_l d$	$\frac{2}{3} bcn = \cdot\dot{6}6 bcn_l d$	$\frac{1}{2}\, bcn = \cdot 5 bcn_l d$
2	T	$tA = tA_l bd$	$tA = tA_l bd$	$tA = tA_l bd$
3	n_l	$\frac{4}{\pi} A_l \frac{t}{c} = 1 \cdot 27 A_l \frac{t}{c}$	$\frac{3}{2} A_l \frac{t}{c} = 1 \cdot 5 A_l \frac{t}{c}$	$2 A_l \frac{t}{c}$
4	$d - a$	$\frac{4n}{3\pi} = \cdot 4244 n_l d$	$\frac{3}{8} n = \cdot 375 n_l d$	$\frac{1}{3} n = \cdot\dot{3}\dot{3} n_l d$
5	a_l	$1 - \cdot 4244 n_l = 1 - \cdot 54 A_l \frac{t}{c}$	$1 - \cdot 375 n_l = 1 - \cdot 5625\, A_l \frac{t}{c}$	$1 - \cdot\dot{3}\dot{3} n_l = 1 - \cdot\dot{6}\dot{6} A_l \frac{t}{c}$
6	minimum value of a_l	$\cdot 5756$	$\cdot 625$	$\cdot\dot{6}\dot{6}$
7	$\frac{R_c}{bd^2}$	$\frac{3}{16} c \pi^2 a_l (1 - a_l) = 1 \cdot 85 c a_l (1 - a_l)$	$\frac{16}{9} c a_l (1 - a_l) = 1 \cdot 7\dot{7} c a_l (1 - a_l)$	$\frac{3}{2} c a_l (1 - a_l) = 1 \cdot 5 c a_l (1 - a_l)$
8	$\frac{R_t}{bd^2}$	$tA_l a_l$	$tA_l a_l$	$tA_l a_l$
9	$\frac{R}{bd^2}$	$tA_l \left(1 - \cdot 54 A_l \frac{t}{c}\right)$	$tA_l \left(1 - \cdot 5625 A_l \frac{t}{c}\right)$	$tA_l \left(1 - \cdot\dot{6}\dot{6} A_l \frac{t}{c}\right)$
10	limiting value of A_l	$\cdot 7854 \frac{c}{t}$	$\cdot\dot{6}\dot{6} \frac{c}{t}$	$\cdot 5 \frac{c}{t}$

The above table was omitted from the May issue ; it should have followed immediately after the symbolic notation at top of page 152.

inforced beams. The author's method is to make sure that the compression steel cannot buckle outwards by providing proper anchors and then to determine the moment of resistance for such steel, as though it were a steel beam, taking its arm as the distance between the top and bottom reinforcement, and add that to the moment of resistance taken by the concrete. Adding the total compression in the concrete to the total compression in the steel, we get a grand total which, divided by the permissible stress, gives the amount of tensile steel to be provided.

There may be convenient opportunity later to go more into detail on this subject.

PILLARS.

We are led next to the question of the strength of reinforced concrete members subjected to simple compression as, for instance, short pillars. Here again the modular ratio comes into customary calculations. Of recent years, however, doubts have been thrown upon the adequacy of such theory more particularly in America where attention has been drawn to the fact that concrete when hardening undergoes contraction, this continuing for a long period of time. This contraction induces a longitudinal shortening in the steel, and may induce so much strain therein that the stress can actually reach the yield point of the steel. That, however, can only apply in the case of a moderate quantity of steel. If a very great quantity of steel be provided, and there be little or no load on the pillar, the concrete would try to diminish the length of the steel bars in contracting, but the steel would be strong enough to hold firmly, with the result that it would crack the concrete, which would be put in tension. The steel would, of course, have suffered a small compression due to the contraction of the pieces of concrete that were still uncracked, so far as adhesion would permit, but such adhesion would be largely destroyed. If now a load be applied, the steel would undergo further compression, and the cracks would be closed. Still further loading would begin to put compression into the concrete which was originally under tension, owing to the steel trying to snap the concrete. In a pillar or beam that is not so heavily reinforced the same argument applies—the contraction of the concrete tends to put compression into the steel, and the steel in turn tends to put tension in the concrete. The load that is taken on the steel must come off the concrete. Therefore, though the steel may be stressed to the yield point the ability of the concrete and steel as a combination will be just the same as

if no contraction had taken place, because the initial compression in the steel resulting from the contraction is only causing initial tension, as one might say, in the concrete which has to be taken out subsequently before we can reach the ordinary compressive resistance of the concrete. Naturally, such contraction of the concrete upsets any ideas we may have about the stresses in the concrete and steel being proportionate to the ratio of their moduli of elasticity. So long as adequate precautions are taken to prevent buckling of the steel we can calculate the resistance quite simply without having anything to do with the modular ratio, and consider it merely as made up by the concrete being able to withstand as much as it would if it were plain concrete and the steel as much as a short steel pillar. The addition of the two will be the total resistance of the reinforced concrete pillar. That is, supposing the steel has an ultimate compressive resistance of 55,000 pds. per square inch, and the concrete 2,000 pds./in.^{2}, and that the pillar contains 100 square inches of concrete and 2 square inches of steel the total load that the pillar will carry is 200,000 pds. plus 110,000 pds. = 310,000 pds. The reinforcing rods being compressed naturally tend to buckle, and the ability to develop the ultimate strength all depends on adequately restraining the rods from buckling. Fig. 38 shows by dotted lines the form that such buckling would take. The links tying these vertical rods together are indicated. If the steel rods and links formed an open structure without any concrete in the middle the rods could buckle inwards, but as it is they are forced, if they are to buckle at all, to go outwards. Years ago the author adopted a standard spacing of links with the object of ensuring that at ultimate rupture the steel could be stressed to its maximum compressive resistance without being liable to buckle as a slender strut. The following gives the calculation for maximum permissible spacing :—

Fig. 38 shows a portion of a reinforced concrete pillar, the rods of which are retained by links from bursting outwards the spacing of which should not exceed 16 times the diameter of the rods if the maximum stress is to be developed therein. W. Alexander's " Columns and Struts " provides precise but practical data for determining the forces developed in latticing of steel pillars, and we may adapt his method to the present case. The links will evidently have to carry the forces that would be developed at the two bearings of pin-ended struts. A portion, so considered, is shown enlarged in Fig. 39.

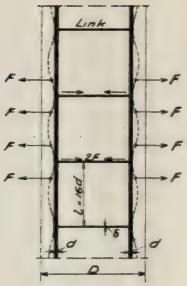


Fig.38

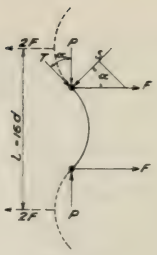


Fig.39

Then $F=\frac{s}{\cos\ a}=\frac{P\ sin\ a}{cos\ a}=\frac{P\ sin\ a}{\sqrt{1-sin^2\ a}}=\frac{Pv}{\sqrt{K^2-v^2}}=\frac{Pv}{K}$ appr.

Therefore force to be developed in a length $16d$

$$=\frac{2Pv}{K}=\frac{2}{n}\sqrt{E\left(f_{max}-f_d\right)}$$

Taking values of a reduced direct stress in rods $f_d=$ 40,000 pds/in^2 and maximum stress (combined bending and direct stress) $f_{max}=60{,}000$ pds/in^2 we get $P=40{,}000\frac{\pi d^2}{4}$, $I=\frac{\pi d^4}{64}$, $n=\frac{d}{2}$, $E=30{,}000{,}000$. Inserting in the formula

$$\text{Force to be developed}=\frac{4}{d}\sqrt{\frac{40{,}000\pi^2 d^6}{4\times 64\times 30{,}000{,}000}}(60{,}000-40{,}000)$$

$$=\frac{10{,}000}{\sqrt{3{,}000}}\pi d^2=573d^2\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

Calling the pitch of the links p

the number of links $n=\frac{16d}{p}$.

An initial stress will be put in the links by lateral swelling of the concrete under the longitudinal compression. Taking Poisson's ratio as $\frac{1}{4}$ and the maximum longitudinal shortening as $\frac{1}{400}$ we ascertain the initial stress to be $\frac{1}{400}\times\frac{1}{4}\times 30{,}000{,}000 = 18{,}750$ pds/in^2. Assuming that we limit the stress in links to the yield point of mild steel, say 40,000 pds/in^2, the maximum stress that can be put in any link will be 21,250 pds/in^2.

Therefore the resistance of each link will be

$$21250\ \frac{\pi\ \delta^2}{4}$$

and of n links $=21250\ \frac{\pi\ \delta^2}{4}\cdot\frac{16d}{p}=267{,}000\ \frac{\delta^2 d}{p}$.....(2)...

Equating (1) and (2)

$$573d^2 = 267{,}000 \frac{\delta^2 d}{p}$$

$$p = 466 \frac{\delta^2}{d}$$

For a $\frac{3}{16}$ inch diameter round link $p = \frac{16 \cdot 4}{d}$

For the smaller bars $16d$. will be the limit, so that we get the pitch for different sizes of bars, say: $\frac{1}{2}$ ϕ 8 ins., $\frac{5}{8}$ ϕ 10 ins., $\frac{3}{4}$ ϕ 12 ins., 1 ϕ 16 ins., $1\frac{1}{4}$ ϕ 13 ins., $1\frac{1}{2}$ ϕ 11 ins., $1\frac{3}{4}$ ϕ 9 ins., 2 ϕ 8 ins. The author adopts as a standard 8 ins. throughout up to 2 ins. diameter rounds.

It will be seen that the links are merely employed to restrain the rods from buckling and are not for the purpose of strengthening the concrete by hoop tension.

There is altogether insufficient experimental data regarding what happens in testing slender reinforced concrete pillars to destruction to be able to give a fully rational method of calculating such members. In view of this it is best not to employ slender pillars, which can be done by keeping down the unsupported length. We must, however, deal with the cases of combined bending and compression in struts, as also compressive reinforcement in members subjected to bending alone. This paper is already too long to elaborately treat the subject. Those who are competent to calculate combined thrust and bending will be quite able to adjust the author's methods, if they approve of them, to the problem. Some opportunity may present itself later for the author to publish his detailed views on this subject.

It may be as well, however, here to refer generally to the matter of monolithic construction where the beams and pillars are rigidly coupled together. Bending in the beams induces bending in the pillars, and vice versa. There has been a good deal written on the subject in recent years about most interesting and valuable investigations, to show what great bending strains may be induced in pillars by the bending of either steel or reinforced concrete beams connected thereto. Determination of the bending moments and shearing forces in the various parts is computed by two mathematical methods, one being to consider the change of slope of the axis of such members when deformed by bending, and the other to employ the theorem of least work. Such essays serve an excellent purpose. The only question the author wants to raise is as to the conclusions that are to be drawn

from such investigations. Too frequently these seem to be that previous methods of designing pillars are all wrong. The bending moment on pillars is treated as being equivalent to applying the direct load eccentrically. Eccentric loading when the eccentricity is actual, as, for instance, a load suspended or lodged on a bracket on the side of a pillar, is very different from an equivalent eccentricity of that kind from a practical point of view. Mr. E. G. Beck, in his book on "Structural Steelwork," has pointed out various erroneous methods of trying to determine the stresses induced by actual eccentric loads, and has demonstrated accurate methods of analysis, but when it comes to equivalent eccentricity induced by the bending of a beam rigidly connected with a pillar great extravagance can be indulged in.

Some of the essayists have suggested that provision must always be made to fully resist the stresses calculated to be due to such equivalent eccentricity, and if one were to follow just what they say, we should have to conclude that a very great number of structures have been erected that must be in imminent danger of collapse, and more than that, they ought to have collapsed. Seeing that they have not makes one desire to find the explanation. At the expense of being considered trite, I would like to remind the exponents of these newer methods that theories should endeavour to explain the facts. The author's view is that the explanation of why pillars could be calculated in the past merely for the direct load, and designers practicaly ignore the bending effects induced by a beam rigidly coupled thereto, and no danger resulted, is merely that plastic deformation came in to assist. Take, for example, a simple portal, consisting of just two pillars sustaining a beam at the top. If the beam were connected to the pillars and the beams happened to be rather shallow, and the pillars rather short and stumpy, the pillars would endeavour to restrain the ends of the beam, with the result that the bending would tend to cause big bending moments in the ends of the beam and the tops of the pillars. Now, if the pillars were not able to sustain such bending moments, they in turn would be bent. As soon as they bent, however, the fixity at the end of the beam would partly give way, and the beam would have to develop more of the curvature it would have had if it had been simply supported. The stresses induced in the pillars would be sufficient to exceed the yield point, and the metal would stretch. But as soon as the pillars had been bent,

and the beam took more of the ordinary curvature of a simply supported beam, there would be no longer the big bending moment put in to the pillars. The two tendencies will naturally give some intermediate value between, that is, the pillars will take as much bending as they can, and the beam will have to take the rest. The bending of the pillar creates an actual eccentricity, but a very small amount. The plastic deformation, therefore, is the means of securing safety in such a case.

FACTORS OF SAFETY.

Euler and succeeding mathematicians who developed the theory of elasticity, strangely enough, never devised a mode of computation whereby working stresses in slender pillars can be estimated. They merely tell us the moment at which a pillar will fail, and leave us to obtain the safe working load, by dividing the ultimate load by a factor of safety. Only in regard to beam design have engineers changed over from the earlier practice of computing ultimate strengths and adopting factors of safety thereon. Seeing how far astray those engineers can wander whose minds are concentrated on trying to calculate working stresses the author prefers to revert to the procedure of some generations of eminent engineers of the past, who based their practice upon experience and upon experiments made under practical conditions on full-sized or model members, and did not depend so much upon mathematical theorising about ideally elastic materials, and elaborate experiments that did not conform to practical conditions. Different factors of safety were adopted for the same material in different circumstances; that is to say, a railway bridge or a machine-part subjected to impact and fatigue, have higher factors of safety than parts of buildings subjected to ordinary static loads.

As a result of the modern interest in the mathematical theory of elasticity great emphasis has been put upon the necessity of keeping within elastic limits or yield points. That is all very well, in the case of ordinary mild steel for riveted structural members, because we have seen that in such cases we need a ductile material, that is one in which the plastic deformation can be considerable. When we come to other materials, such as concrete and reinforced concrete, such very different conditions apply that we need to view the matter from a different angle to that of the elasticians who, in order that their mathematical treatment should apply ask for as elastic material as possible. In the case of mild steel we may contend that it comes to the same

thing whether we divide the yield point by, say, a factor of two, or the ultimate strength by a factor of four, in order to obtain a working stress, that is to say, to all intents and purposes the same result would be obtained whether we calculated the ultimate strength and divided it by four, to obtain the working load, or whether we took the working load and made a calculation from it to keep within the stress derived by dividing the ultimate stress by a factor. In the case of materials which are subject to much plastic deformation, however, the mathematicians' theory for elastic materials does not strictly apply, and should not be used as an argument in favour of the working stress method of elasticians. We can quite well figure the resistance in terms of any stress we like, so as to give the working load straight away. The author desires to emphasise that such a stress must be derived from the ultimate stress that is estimated to be realised by loading to destruction, and is not the same as the actual stress at working loads which has become known as the working stress.

An engineer has to get on with the job, and make a practical solution with the best assistance he can obtain from the work of those engaged in scientific research. No doubt in time we shall be enlightened much more as to the true nature and constitution of structural materials, and as to such matters as fatigue, and in the light of such greater knowledge we may be able to reduce our factors of safety, but it is rather too much to ask us as a result of some mathematical analysis to declare that all the engineers of the past have not built thoroughly sound and safe structures. The path of progress in engineering is to employ the best scientific knowledge to economise in material and labour, i.e., conserve natural resources. The ancient Egyptians and the Greeks with an elementary knowledge of building erected safe buildings at the expense of a great amount of slave labour and huge masses of material. Gradual progress has cut down the quantity of material required, and saved an enormous amount. Strangely enough though there is a greater refinement to-day in structural engineering calculations than ever before the same old factors of safety are used to determine working stresses as were adopted by earlier practitioners who employed much simpler methods of calculation.

ERRATA.

Page 150, line 40—a should be a_1 = arm depth ratio.
Page 160, last line—insert "to" before "the lower."

DISCUSSION.

Dr. Oscar Faber, O.B.E., D.Sc., Assoc.M.Inst.C.E., &c. (Member) said the author had dealt with many subjects, but in his opinion, his real points would not bear discussion for a moment. With regard to plasticity, for example, that mild steel and other materials had their yield point was no new thing, and yet the straight line law had been used invariably in calculations under working stresses. The lecturer had referred to monolithic structure calculations, and had said that although they had found those calculations very useful it must be remembered that the materials of which practical structures were composed were not ideally elastic bodies. They had remembered those things, but they were designing structures to act within working stresses, and within working stresses the structures they were designing were practically ideally elastic bodies; both concrete and steel, within working stresses, were within 1 per cent. ideally elastic bodies, and if calculations and formulæ could be got accurate within 1 per cent., they were doing very well. He hoped to show that Mr. Dyson's formulæ were not within 100 per cent. in some places. The author had said that the ordinary formula for the strength of reinforced concrete was empirical. He (the speaker) did not agree. The straight-line law had been tested over and over again by experiments made with extensometers, and it had been found that anywhere within working stresses, and considerably outside working stresses, the beams behaved in close agreement with theoretical assumptions. It was true that in very short beams, where the shear stresses were great in comparison with the bending stresses, these laws were not so accurately followed, but for beams of ordinary ratios of length to depth they were very accurate indeed.

The lecturer had made a great point of Talbot's beams, and had pointed out that of two particular beams which had the same amount of steel, in the one case soft and in the other hard, one carried a maximum load of 26,000 lb. and the other 34,300 lb., and asked how that difference was accounted for. As a matter of fact, these were isolated beams, and with the particular concrete he was using, which was particularly poor—1 cement, 3 sand and 6 stone—considerable variations were obtained from isolated tests. In his opinion, that was quite sufficient to explain the disparity between the two results. Certainly on that kind of evidence he would not be prepared to overthrow the years of work which had resulted in bringing modern theorists where they were and take up a totally fresh theory for which there was no scientific basis whatever, except that the lecturer assumed an elliptical stress distribution above the neutral axis,

and let the neutral axis go just were it liked. All the tests the lecturer had referred to were tests on rectangular beams with very high percentages of reinforcement—2·76 per cent., and so on. Beams with those percentages were never used in practice, and if the lecturer could not find something in support of his theory except in structures where the percentages were 2·76, for example, about eight times as high as were used in practice, then he did not think the lecturer's theories needed very serious consideration for application to practical structures.

The real point in the paper was the fresh theory for rectangular beams, singly reinforced, and the theory was that the steel was stressed to 16,000 lb. per sq. in. and the concrete to 600 lb. per sq. in., whatever the percentage of the steel. Tests on the subject showed that the ratio between the steel stress and the concrete stress depended on the percentage, whereas in the lecturer's theory it did not. The lecturer indicated that his stress line " had to be stopped when a percentage was reached that got the maximum compressive resistance out of the concrete. Thus, it showed the point at which, if any more steel were inserted, it would give no greater strength—a fact with which most reinforced concrete specialists were familiar." He (the speaker) did not think it was a fact, but that it was wrong. So far as he knew, the more steel that was added the greater was the strength of the beam. After a certain point the additional strength obtained was very small in comparison with the additional steel put in, and certainly a point was reached where it did not pay to use more steel, but nevertheless, if more steel were added certainly more strength was obtained.

With regard to stress distribution, the lecturer considered a stress distribution in which the concrete stress came right down to the steel.

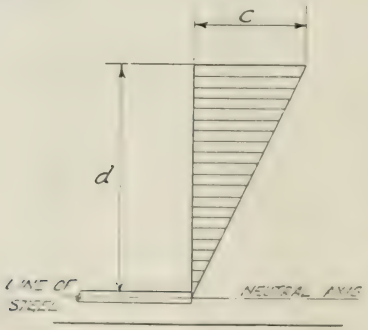


Fig 1

When dealing with heavily-reinforced beams the lecturer's stress distribution in the case of the strongest beams was as indicated in *Fig. 1* ; in other words, the point where the concrete stress diagram cut the axis was at the level of the steel. But the point where the concrete stress diagram cut the axis was what was usually called the neutral axis, and was the point in the beam where the stress was zero. Yet it was at that very point the lecturer assumed the steel to be stressed to 16,000 lb. per sq. in. ; in other words, at that particular layer, when considering the concrete, he took zero stress which involved zero deformation while at exactly the same level he assumed that particular layer in the beam to be stretched sufficiently to produce a stretch of 16,000 lb. per sq. in. in the steel. Those results were absolutely inconsistent and that stress distribution was an impossible one.

The lecturer gave a table showing slabs designed on the ordinary theory and his own theory, and showed that with the ordinary theory a slab not more than $5\frac{1}{2}$ in. thick would carry a certain load which could be determined within $12\frac{1}{2}$ per cent., and with his own theory, $1\frac{1}{2}$ per cent., which was very useful. But, as a matter of fact, if the thickness of the slab were increased by about $\frac{1}{2}$ in. it would be found that the ordinary theory would also work out to about $1\frac{1}{2}$ per cent., and would in addition have theoretical and experimental justification. The lecturer had also stated that his theory gave a factor of safety of 4, but did not prove it. The ordinary curve between resisting moment and percentage was a curve which went up more or less straight to a certain point, and then took a curved form.

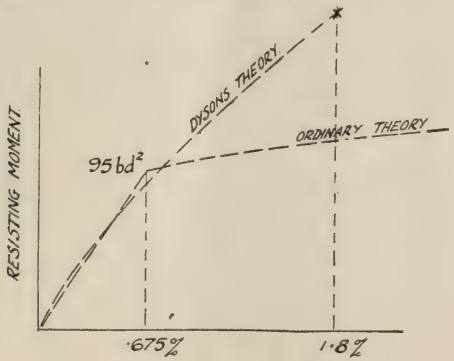


Fig. 2

He (the speaker) suggested that a curve which gradually increased at a lessening slope with increased percentage was

much more likely to be in accordance with facts than a curve which went up to 1.8 per cent. and then went off horizontally. As to factor of safety, in comparing the ordinary theory with actual tests, he had published some tests of his own research work on rectangular slabs some years ago, both singly and doubly reinforced, which gave the following results:—

RESULTS OF EXPERIMENTS OF SINGLY REINFORCED BEAMS.

Percentage of steel.	Safe load calculated by ordinary safety.	The breaking load obtained by experiment.	Factor of safety equals breaking load divided by safe load.
Zero	428 lb.	1,650 lb.	3·8
0·234	1,120 ,,	5,330 ,,	4·7
·467	2,180 ,,	7,950 ,,	3·6
·73	3,180 ,,	13,350 ,,	4·2
1·05	3,530 ,,	14,450 ,,	4·1
1·43	3,900 ,,	18,050 ,,	4·6
1·86	4,230 ,,	19,150 ,,	4·5
2·37	4,450 ,,	21,950 ,,	4·9

It would be seen from the table that over a very wide range of percentages, namely, from 0 up to 2·37, the ordinary theory gave factors of safety which were remarkably consistent and of the right order, namely, 4, since the stresses forming the basis of the calculations for the safe load were taken at one-quarter the ultimate strength of the materials. The lecturer's formula gave very much higher results for all percentages higher than about ·7, and as the experiment showed that the ordinary theory was right it followed that the lecturer's theory, which gave very much higher results, must be wrong.

RESULTS OF EXPERIMENTS OF DOUBLY REINFORCED BEAMS.

Percentage of steel.	Safe load calculated by ordinary theory.	Breaking load obtained by experiment.	Factor of safety equals breaking load divided by safe load.
·467	2,220 lb.	10,600 lb.	4·8
·73	3,330 ,,	12,950 ,,	3·9
1·05	4,620 ,,	18,180 ,,	4·0
1·43	5,500 ,,	20,450 ,,	3·7
1·87	6,400 ,,	31,450 ,,	4·9
2·38	7,400 ,,	33,680 ,,	4·5
2·9	8,400 ,,	41,080 ,,	4·9
3·56	9,600 ,,	38,950 ,,	4·1

The safe loads in the above calculations were obtained by putting the stress in the compression of steel as the stress in the concrete at that distance from the neutral axis multiplied by the modular ratio 15, and it would be seen that over a very wide range of percentages the factor of safety remained practically constant and of the right order throughout the whole range, which was very strong justification indeed of the substantial accuracy of the assumptions made. Here again, it follows that, as the lecturer's theories admittedly gave a quite different result and the experiments showed that the ordinary theory gave substantially accurate results, it followed that the lecturer's theories were not supported by the experimental evidence. The beams were made by hand, and variations were, therefore, likely to occur, but, subject to those necessary variations the factors of safety were sufficiently uniform, and averaged 4 as near as averages could be expected to go. The compressive strength of the concrete was about 2,400 lb. per sq. in.

The lecturer had pointed out that there was such a thing as shrinkage and yielding of the concrete. That was true, but the modular ratio still had a value. They could make allowance for those two effects. They had the effect of throwing stress on to the steel and taking it off the concrete, but the modular ratio was still of value in the determining of stresses.

The lecturer had said " If the pillars are not able to sustain such a bending moment they in turn will be bent." He (the speaker) pointed out that they could not bend a structure without applying bending moment to it, so that the mere fact that " they in turn will be bent," implied that there would be a bending moment in the pillar. Then, the lecturer said " The stresses induced in the pillars will be sufficient to exceed the yield point, and the metal will stretch." How could they exceed the yield point without very high stresses, which was what they were trying to avoid? " But as soon as the pillars have been bent, and the beam takes its normal curve, there will be no longer the big bending moment put into the pillars." Those statements would really not bear thinking about. So soon as they started to try to prove that there was no bending in pillars they would invariably tie themselves into knots, and the lecturer, he claimed had done so. Talking of calculations derived from people who worked out bending moments of pillars, the lecturer had said " Too frequently these seem to be that previous methods of designing pillars are all wrong." This, by the way, was curious comment from the reader of a paper which set out to prove that the usual method of designing slabs was all wrong. What they who were interested in calculating the moments in pillars try to do was to differentiate between those

pillars which would have heavy bending stresses and those which would have small bending stresses, and make a differentiation in those columns in actual practice. As a summary of his criticism, he claimed that the ordinary theory gave substantially accurate results, both for slabs and reinforced concrete beams, that no case had been put forward that the lecturer's new theory gave more accurate results, and, in point of fact, the author's results led to all sorts of fallacies and absurdities.

Referring to the author's Figure 28, it can be shown quite easily that the stress distribution would be quite different from that given, and would be compression at the top dying down to the neutral axis (some half-way down) and tension below.

Mr. Charles F. Marsh, M.Inst.C.E. (Member), said the title of the paper should have been "Why Reinforced Concrete?" The author appeared to be obsessed with the ultimate effects, and deprecated the customary methods of basing calculations on working stresses. Much happened between conditions under which a member actually functioned and the conditions under ultimate stresses. Ultimate conditions were in no way applicable to working conditions, and that was universally recognised in the design of all structures. For one reason, the concrete above the steel did not crack in properly designed members subjected to flexure under working load. The elongation at the axis of the steel when the metal was stressed at 16,000 lb. per square inch, was about 0·53/1,000ths, and with a high tensile steel stressed at 20,000 lb. per square inch, it was about 0·66/1,000ths. Experiments had shown that concrete reinforced with steel would suffer greater strains before cracking. Below the steel the strain would naturally increase, and it would probably crack, at any rate, when high tensile steel was used unless a comparatively low limit was adopted for the tensile resistance. Generally, with respect to the modulus of elasticity, the elasticity of materials is a fact which ought not to be neglected in design. The laws of elasticity must apply to reinforced concrete as to any other structural material, as, after all is said and done, the fact remains that the strain on the concrete is greater than that on the steel under the same stress. As simplifying the necessary calculations, however, he was firmly convinced that the value of 15 for the modular ratio was sufficiently accurate for all calculations. The reasons for this he had fully set out in the paper he had read before the Institute on the L.C.C. Regulations. The author had very much criticised the customary methods of calculation, and stated that the reason for doing so was that the hypotheses used were not absolutely correct. Most of the

points he had put forward were dealt with in a book published by the speaker 18 years ago, which dealt directly with the same things which were brought forward by Mr. Dyson. When he, Mr. Marsh, dealt with the hypotheses, he had made this statement: — "It must, however, be borne in mind, that all theories, even the most elaborate, are only approximate, being based on the best information that can be obtained on the subject. We are, consequently, justified in assuming hypotheses on which to base our calculations, which may not be absolutely correct." All engineering calculations were based on hypotheses, and all these hypotheses were to some extent untrue: the calculations for reinforced concrete were just as true as the calculations used for other structural designs. As to the plane sections remaining plane after bending, he had said in his book "We must treat this hypothesis with suspicion, as at best it is only an approximation to the real state of the case." That hypothesis was no more true for a steel joist than for a reinforced concrete beam, but it was sufficiently accurate for the purposes of working calculations. As to the factor of safety, that covered many doubtful matters other than the actual resistance and elasticity of a material, and if they did know everything about the materials, and could be assured of perfect workmanship, they might possibly do away with the factor of safety altogether. There was no doubt that the author had got rather an idealistic mind and, after studying the author's new method of calculation as set out in the paper, it seemed to him that if it were generally adopted some very dangerous structures would be erected.

Mr. Ewart S. Andrews, B.Sc (Eng.), (Member), in seconding a vote of thanks to the author, paid a tribute to him for the very great skill with which he had written it, and for the great amount of work he had put into the preparation of the diagrams. The author had started the paper by taking his hearers a conducted tour through the whole realm of elasticity, and by the time they were all dazzled he introduced them to the real part of the paper, which dealt with the modular ratio. Mr. Andrews agreed with the main conclusions of Dr. Faber, that the evidence given in the paper did not prove the uselessness of the existing theories.

With regard to brittle material, it did not follow that high tension steel was necessarily brittle; it did not necessarily follow that, because the elastic limit of steel was not far from its ultimate strength, it was a material quite unsuitable for structural work, and so far he agreed with the author. It was in piecing together the variations from accepted fact

that he could not agree. As to the compressed area extending down as far as they liked, clearly it could not extend below the cracks, which they knew, from experiment, existed. With regard to Talbot's experiments, figures were given for the position of the neutral axis, and from a very rough calculation he had just made, it seemed to him that the calculation, working backwards from Talbot's results, did not differ seriously from the ordinary method of calculation.

The old point was raised in the paper as to whether they were to base their calculations on working stresses or ultimate stresses, and this question could not easily be settled. They had been told that the ultimate standard was the testing machine, and he strongly urged that that was rather a dangerous doctrine. People who were out to prove all sorts of illogical theories could usually give results of some tests, but, in addition to making tests, they had to ensure that they were made by skilled people, and they had to be sure that the results were interpreted by skilled people.

The PRESIDENT did not think the doctrine was dangerous, but a very safe one, but he agreed that there was a danger of error in interpreting the results of tests. He agreed that the testing machine must be in the hands of the right person.

Mr. HARRINGTON HUDSON (Member), said that many points which he had intended to raise had already been discussed by Dr. Faber, so his contribution to the discussion would be brief. In speaking of reinforced concrete subjected to simple compression, the author had referred to the gradual contraction of the concrete. On page 144 he stated "such contraction of the concrete upsets any ideas we may have about the stresses in the concrete and steel being proportionate to the ratio of their moduli of elasticity," and then stated that, provided buckling of the rods could be prevented, the resistance of a reinforced concrete pillar was the sum of the resistances of the plain concrete pillar and of the steel. That seemed rather a sweeping statement. He agreed with the author that the present-day formulæ for design of pillars might well be revised in the light of recent investigations with regard to contraction, but he could not imagine that the design could simplify itself to the extent suggested in the paper. This contraction was divided into two parts, one, the shrinkage of concrete, and the other the plastic yield, and they should be considered separately. Shrinkage had been investigated in 1916 by A. C. Janni, who showed that in ordinary concrete used in reinforced concrete work, the shrinkage approximated to ·05 per cent. in two or three months, and then remained fairly constant. Plastic yield was rather a doubtful factor. Shrinkage had quite a considerable effect upon the stresses in a pillar. It was rather

interesting to see how much stress had to be put into the steel before the strength of the concrete could be brought into play ; he had worked out the value, and, taking 30,000,000 lbs./in^2 as the elastic modulus of steel, and multiplying it by 0·0005 this gave the actual compressive stress of the steel when the initial tensile stress in the concrete was reduced to zero. That worked out to 15,000 pnd. per sq. inch. Then, working the concrete up to, say, 600 pnd. per square inch would bring an additional 9,000 pnd. per sq. inch on to the vertical reinforcement, and the final stress in the pillar would be 600 pnd. per sq. inch in the concrete and 24,000 pnd. per sq. inch in the vertical reinforcement.

Mr. G. N. Gardner (Member) expressed the hope that more attention might be devoted to one of the most important points raised by the author, namely, the modular ratio. The author had discarded the ordinary theories, owing to the fact that cracks developed, and he had said that the shear could not be communicated, owing to these cracks, but incidentally he gave no information with regard to the distribution of cracks. Then the author had also remarked that the more the beam was stressed the lower the compression area would go, until it reached the steel. The inference to be drawn from this would be that the greater the deflection of the beam the greater the tendency of the cracks to close! Was that so? Would the author give the diagram of the truss action in the case of a thin slab of 20 ft. span reinforced in tension only.

The President, dealing with two points of history, said that the author had stated that M. Edmond Coignet, together with his engineer, M. N. de Tedesco, were the first to suggest introducing the modulus of elasticity of the concrete and of the steel into the calculations. As a matter of fact, they only introduced that into the calculations for reinforced concrete because, in this country, a generation before that, railway engineers employed the modular ratio in ascertaining the strength of wooden flitch beams, in which they had to take into account the elastic modulus of the steel flitch and the elastic modulus of the wooden flanking pieces. Mons. N. de Tedesco applied that method to reinforced concrete.

The author had also said that the German professors, Mörsch and Bach, made notable contributions, from whose experiments a justifying coefficient was chosen to make the formulæ somewhat fit the experiments, " the said coefficient being termed the modular ratio."

He (the President) suggested that the paper should say " the said coefficient being now termed the modular

ratio," because he claimed that he himself had introduced the term "*modular ratio*," which was now accepted through out the English speaking world as the short name for the ratio between the elastic modulus of the steel (E_s) and the elastic modulus of the concrete (E_c).

Mr. J. S. E. DE VESIAN, M.I.Mech.E., M.S.E. (Member) sends the following written contribution:—

In his very interesting paper Mr. Kempton Dyson throws doubts upon the correctness of the generally accepted theory that the so-called modular ratio is the fundamental factor.

I quite agree with him when he states that the computation of stresses subject to plastic deformation has not received sufficient attention, seeing that all concretes are neither perfectly elastic nor soft. It is not possible to make precise laws governing the deflection of concrete, the properties of which depend upon so many factors, such as the proportion and nature of the inert matters mixed with the cement, the quantity of water used, the intensity of ramming, the age of the concrete, the atmospheric conditions when the cement is setting, etc. Moreover, there are reasons to believe that the coefficient of elasticity of a given concrete may vary with the external stresses to which it is subjected. The uncertainty of the position of the neutral axis together with a relative plasticity of concrete are two causes of error in the calculation based upon a fixed modular ratio. As a matter of fact, the method advocated by Mr. Dyson has many points of similarity with the semi-empirical methods applied with success by the pioneers of reinforced concrete.

We must remember the numerous floor slabs (Monier) made so widely in Europe, and the Hennebique constructions built all over the world for the last 28 years. The calculations of these structures were not based upon the principle of the modular ratio. I may add that the columns designed by my firm were formerly calculated in practically the same manner as in Mr. Dyson's method. The New General Post Office is a good example.

Even at the present day most continental constructions on the Hennebique system are calculated with the same semi-empirical formulæ proved and adopted by Mr. Hennebique 28 years ago. I have inspected a large number of these structures, and I have found them as free from defects in every way, and as sound as buildings calculated with (so-called) scientific methods

I desire to congratulate Mr. Dyson upon his paper, which, while refuting some "very elastic" if generally accepted principles, boldly proposes a method which, in my opinion, is based upon sounder principles.

The theory enunciated certainly fits with observed results better than the modular ratio method seems to do. Mr. Dyson points this out in quoting Sir Benjamin Baker's work of 1870, and others. The elastic theory does not adequately explain why loading far in excess of the theoretical ultimate can be imposed upon sections without resulting in destruction. In fact this very fact is in itself a demonstration of some inherent fallacy in the elastic theory.

The following figures worked in accordance with the formulæ devised by Mr. Hennebique before the adoption of the modular ratio, and still used on the Continent to a very large extent, will, I think, form an interesting addition to and comparison with the figures given by Mr. Dyson in his table on p. 160, giving the steel required for slabs of varying thickness:—

Thickness of slab	8″	7½″	7″	6½″	6″	5½″
Percentage A_s	0·55	0·66	0·80	1·04	1·30	2·00

Mr. Gower B. R. Pimm, Assoc.M.Inst. C.E. (Member), sends the following written contribution:—

Mr. H. Kempton Dyson's paper will probably divide designers of reinforced-concrete into two camps—those who realize that a big obstacle has been overcome, and those who feel that a breach has been made. But whatever may be said for or against his theory it must be welcomed as a healthy stimulus.

There is a school of designers (and a fairly large one), who are altogether too academic, and whose point of view finds expression in such matters as minute variations of dimensions in different parts of a structure, when, for practical reasons, it would often be truer economy to adopt the maximum sizes throughout; and more broadly in the disregard of such matters as the human element, general fitness, and even the most obvious local conditions. Not that failure to have regard to the last-mentioned is always the fault of the designer, for under the present pernicious system of competitive design he often has no opportunity of making himself acquainted with local conditions, and consequently he has to make "assumptions" with regard to matters which, under a proper system, would be the subject of definite investigation. For example, many of the reinforced-concrete structures around the coast have quite obviously been designed without regard

to any considerations whatever except vertical loading, their ability to resist the impact of waves and ships being simply that which they happen to possess when designed as if they were land structures. In such cases the fact that under the present "modular ratio" method of design, lightness and cheapness do not go hand in hand, does, in many cases, prevent an extravagantly light design from being put forward, but under Mr. Dyson's theory this valuable safeguard would be to a great extent lacking.

The writer expresses these views in the light of certain knowledge of the facts in connection with existing works, and in at least one case it has fallen to his lot to substitute for one of these flimsy structures something more in keeping with the purpose for which it was intended.

That such considerations should be allowed to discount the value of Mr. Dyson's paper would be a matter for the greatest regret, and the soundness of his theory is in no way impaired by the fact that in the hands of any but an experienced designer it would lead to disastrous results.

Mr. Edward Godfrey, of Pittsburgh, Pa., sends a written contribution, of which the following is a summary:—

The galley proofs of a paper by Mr. H. Kempton Dyson, on the subject, "What is the Use of the Modular Ratio?" came to me a few days ago. The subject is so interesting, and the paper touches on so many things that have engaged my attention in recent years that I should like to have the privilege of discussing the paper.

In my book, "Concrete," which was published in 1908, I advocated the ignoring altogether of the modulus of elasticity of concrete in computing the strength of a beam or slab, for the reason that the value of that modulus is as variable and uncertain as the guesses on the age of the world that scientists amuse themselves by making. Mr. Dyson states that his theory leads to one conclusion, namely, that the modular ratio of the concrete and the steel can be consigned to limbo.

Tests show the neutral axis to be at or near the middle of the depth of the concrete beam. If this position, or some definite ratio of the depth be agreed upon as fixing the position of the neutral axis of a beam or slab, an enormous amount of useless figuring is swept away, and men can give themselves to questions that have a real bearing on the safety of structures. There are many such questions that are of infinitely more importance than the relative factor of

safety of reinforced concrete beams, for that is all that the endless theoretical discussion ever can, or ever will, amount to.

On the much older subject of columns miles of theoretical discussion and myriads of formulas have been published in an apparent effort to arrive at the exact strength of a column, and in nearly all of it the one factor of greatest magnitude is entirely ignored, namely, the manufacture or fabrication of a column. All theoretical treatments of columns, with scarcely any exception, start with a perfect and perfectly elastic column, whereas imperfection in columns as built is very common, and has a very large influence in the strength.

The approximate ultimate strength of a commercially made column is all that theory can ever discover, even if that theory is aided by tests. As to the safe strength, the value arrived at is still more approximate, as it is influenced very largely by judgment. Exactly the same things can be said of reinforced concrete beams and slabs.

The strength of steel tension members can be closely determined, but even here the size of the member has much to do with the character of the steel and with the unit tensile strength. The steel tension member is generally considered to be the type of member which has the most definite or definitely known ultimate strength. But in tension members it is in reality the elastic limit that governs the safe value, and the factor of safety should be based on the elastic value of the member. A factor of safety is partly to cover imperfections in the member or irregularities in the strength and partly to cover uncertainties in the calculated stress.

The slender column comes next to the tension member in the known definite value of its ultimate strength. In fact it is perhaps more definitely known than the tension member, in many cases, if the column be sufficiently slender. The only property of the steel needed to arrive at the ultimate strength of a slender column is the modulus of elasticity, and this is nearly constant for steels, being practically the same for all grades of steel. This is one of the early discoveries in modern structural engineering principles, and yet, as Mr. Dyson points out, Euler himself, who gave us the formula for the ultimate strength of slender columns, never intimated what the working stress in a slender column should be. European engineers use the Euler formula for structurally permissible columns, and specify a factor of

safety of five. That formula has no application whatever to the shorter columns required for structures, for it gives absurdly high values on account of the fact that the elasticity of the steel is passed before the formula value of the ultimate strength is reached. The value in the formula has therefore no relation whatever to the strength of the column, since the loss of elasticity of the column takes it out of the province of the formula, which presupposes elasticity. Furthermore, the use of a factor of safety of five for columns where the Euler formula has application (slender columns) is a waste of material. The Euler formula gives in reality the elastic strength of a column, and the metal has low compressive stress up to practically the point of failure. At that point the column buckles, and continues to buckle under a practically constant load, and it is only after that buckling process that the unit stress on the extreme fibre becomes excessive.

In the slender column loaded to one-fifth of the Euler load the unit stress may be as low as 1/50 or less of the elastic limit of the steel. The question of fatigue or of high stress does not enter in any measure whatever in a really slender column, hence a factor of safety of about 1½ or 2 is all that is necessary.

In the case of shorter columns, and in the case of reinforced concrete beams, slabs, and columns, the predicted strength based on theory may be very far from the actual strength as determined by test, and the erratic results of tests is proof enough that this will always be the case, no matter how refined theory may grow to be. Hence there is a lot of waste of time in evolving finer and finer theories for these things. Imperfections of workmanship have little effect on tension members and slender columns, but they have a large influence on shorter, that is, structurally permissible columns and on reinforced concrete parts. Hence a liberal factor of safety is of infinitely greater benefit than the finest theory imaginable.

In my book, "Steel Designing," a formula is derived for columns which is based on the assumption of imperfection—an imperfection which amounts to an initial bow in the axis of the column with a versed sine 1/300 of the column length. With only this assumption and elasticity in the material a formula is evolved which gives the axial load which will produce an extreme fibre stress of 16,000 lb. per sq. in. The locus of this formula, in structurally permissible columns, corresponds very closely with the usual

straight-line formula used almost universally for columns in America. This formula and the rough agreement of tests of commercially made columns with the straight-line formula is ample justification for the straight-line formula. The formula does not aim to discover the exact safe or ultimate strength of a column, but merely a value for the safe strength that may be expected if usual care is taken in the fabrication.

A simple theoretical value for the strength of a reinforced concrete beam or slab of a given type and quality of concrete can be worked out along similar lines, and only a simple formula is justified where the properties of the material are as variable as those of concrete The simpler the derivation of the formula the more appropriate will be its use. Largely for this reason straight-line variation of the stress in the cross section of a reinforced concrete beam is the best assumption, because it is the simplest and most workable. By locating the neutral axis a given fraction of the depth from the top of the beam, other than straight-line variation, could be compensated for.

Figures 10 and 12 show real shear reinforcement, but I would turn the end hooks the other way, so that the rods would lie near the top of beam at supports, or else, of course, where construction permits, pass the rods into the next beam for anchorage. This kind of shear reinforcement was never permitted in American standards until the last year or two. I have been fighting for its recognition and the exclusion of short shear members for fifteen years (it is not patented.) Recently I published (in "Concrete," of Detroit, Mich., February, 1921) a list of 29 large reinforced concrete failures, principally in America. Several of these failures were the direct result of dependence on short shear members. The beams broke off between the last shear member and the support. Two of the failures were bridge spans that had been in use for years; one was carrying no load when it failed. All of these failures were of types of design that I have been condemning since 1906.

I am in agreement with the parts of Mr. Dyson's paper where he calls attention to the importance of plastic deformation of metals, and shows that assumption of perfect elasticity, as of a steel girder and a column rigidly connected, would mean that there are countless cases where failure ought to be expected. These facts demonstrate what I have frequently pointed out, namely, that secondary stresses in steel work have practically no meaning, and a lot of fine theory about the same belongs in the discard.

This plasticity or toughness is a marked characteristic of steel work, but it has proven to be the reverse in reinforced concrete. This is particularly true of the kind of column shown in Mr. Dyson's Fig. 38. This I call a rodded column as distinguished from a true reinforced concrete column, which has a spiral or hooping instead of the square ties. This column is unsuitable for anything but short, chunky props with balanced loads. In my list of 29 failures a large number of the structures had columns of this sort. One large building was being finished up for occupancy in a few days. It collapsed in utter ruin. In some of the examples the columns failed at loads as low as 150 lb. per sq. in.

As a member of the Joint Committee in 1916 I refused to sign the Report until given the privilege of incorporating a dissenting note rejecting everything in the Report concerning short shear members and rodded columns. Every few months there is a demonstration of the correctness of my position in a large reinforced concrete failure.

Capt. O. Olliff Lee, Assoc.M.Inst.C.E., &c. (Member) sends the following written contribution:—I listened with great interest to Mr. Kempton Dyson's paper, and to the discussion thereon. I feel, however, that out of the heresies put forward by Mr. Dyson, the more important question of "What modular ratio should we use?" would have been evolved if the Members of the Institute had been able to study the paper carefully before the meeting, or had heard the paper read in its entirety, and had had time at the meeting to join issue with the author and be fully answered by him.

I think the author has fallen into the very pitfall you, Mr. President, warned us against, viz., misinterpretation of experimental results.

For example, on the strength of figures 24 to 27, the author evolves figures 28 and 29.

The author admits that steel in tension increases in length and concrete in compression shortens, and in both cases these changes in length commence from the incipience of the stress.

In Fig. 28, it is therefore obvious that the steel bar, from nut round the wheel to the other nut, increases in length when the steel is stressed in tension. How, then, can the author assert that the concrete along the line of the bar and between the shoes shortens in length?

If the concrete does not shorten in length, it cannot be in compression along the line of the bar, and hence the distribution of stress shown in Fig. 28 is wrong.

Again in Fig. 29, the concrete must either increase in length along the line of the bar, by the same amount as the bar increases, or it must crack when deflection takes place.

In my opinion the valuable portion of the author's paper is on page 2, where he quotes Professor Lea, and quotes the use of the " average modulus." There I agree with him.

REPLY OF THE AUTHOR.

Mr. Dyson's written reply to the discussion is as follows :—

The paper and diagrams took a long while to prepare and dealt with an aspect of the subject that has been investigated by the author for years. Naturally it must take anyone some time to see properly all its bearings. It is hoped that it will be studied in detail. The ordinary theory has been criticised—it is believed destructively—and the endeavour has been made to replace it by another theory which it is claimed is supported by facts. If the new theory sufficiently explains and co-ordinates the facts, it is a workable theory. It was evident from the first that there would be prejudice to be faced, but the author feels confident his main contentions must be accepted eventually.

Dr. Faber appears to run entirely counter to the author's arguments, but his criticism shows that he was under some misapprehension as to the author's contentions. For instance, he says : " If the lecturer could not find something in support of his theory except in structures where the percentages were 2.76, for example, about eight times as high as were used in practice, then he did not think the lecturer's theories needed very serious consideration for application to practical structures." It was thought it had been made quite clear in the paper that the new theory gave identically the same result as the ordinary theory for the critical percentage, and practically similar values for smaller percentages, and that it was only in the higher percentages that the divergence in the theories became very noticeable. Obviously, therefore, it was no use citing experiments on small percentages of steel in favour of one theory or the other. If any theory is to hold sway for some time it surely ought to embrace extremes. That is why the author picked out experiments on beams reinforced with large percentages of steel. Dr. Faber's statement that 2.76 per cent. is about eight times as high as used in practice is surprising. Most modern designers use between .6 and 1 per cent., so that Dr. Faber ought to have halved his figure. But some of the pioneers like M. Hennebique, who were particularly practi-

cal, and had a great deal of experience, used very much higher percentages. It is pointed out in the paper that the new theory accounts for what they found to be of practical advantage, that is to say, on the new theory high percentages in slabs are economical (vide pages 159 and 160).

On the question of the straight line law, the author has indicated in a note at the bottom of the table on page 160 that if a straight line or triangular stress distribution be assumed with a higher stress in the concrete, very similar values will be obtained, as by assuming an elliptical stress distribution with a lower and truer value for the ultimate stress developable in the concrete. In the same note it is pointed out that the true crushing resistance ought to be determined on prisms or cylinders whose height is twice the width, and that the strength thus ascertained will be only about 80 per cent. of that given by cubes which are the usual form of test piece. For further explanation of this, the author would refer to his former paper on " Shear." The diagram contrasting the theories (Fig. 32 on page 151) therefore forms a fairly accurate comparison of what the author gets different from the ordinary theory, although he believes that it would be truer to adopt an elliptical stress distribution with lower working values than are usually adopted.

Dr. Faber says, " he hoped to show that Mr. Dyson's formulæ were not within 100 per cent. in some places," but if reference is made to Fig. 32 on page 151 it will be seen that the maximum divergence (assuming the usually allowed stresses) is that the author would increase the calculated moment of resistance by just over 50 per cent. It is suggested, therefore, that Dr. Faber has exaggerated the difference.

The author quite agrees that the quoted tests of Professor Talbot do not finally prove anything; but Dr. Faber seems to waive airily on one side a difference of 32 per cent. as of no account.

Figure 32 (to which Dr. Faber's Fig 2 is similar) shows that according to the new theory, with increasing large percentages of steel the difference between the computed moments of resistance likewise increases. This means that if the ultimate resistances realised show factors of safety in regard to the ordinary theory rising with increasing percentages, the new theory is in closer agreement with the facts. Now, if reference be made to the results of Dr. Faber's experiments on singly reinforced beams, it will be seen that his tests support the new theory in this way.

These experiments of Dr. Faber, the author finds, were

published in the issues of " Concrete " for May and June, 1916. Unfortunately they are no more conclusive than those of Professor Talbot; because though the kinds of materials used are stated, no tests are given as to the real strengths of the concrete and steel employed, and only one beam of each kind was tested. By either theory computations can be made to agree with the experimental results if we allow a possible range of stress. Between the lowest and highest factors of safety computed by Dr. Faber there is a difference of 35 per cent.

Let us see if Dr. Faber's tests can be reconciled with the new theory. Firstly, the comment is made that in commercial mild steel both the yield point and ultimate strength of large diameter bars are less than for smaller bars, because in the smaller sizes the metal cools in the rolling and the finishing is done at a lower temperature, thus putting cold work into the steel. The outside of such small bars is often harder and stronger than the core, as can be found by comparing turned-down test specimens against the rolled section. The size of the bars employed by Dr. Faber in his tests were as follows:—Beam 2, one $\frac{1}{2}$in.; beam 3 two $\frac{1}{2}$ in.; beam 4, two $\frac{5}{8}$ in.; beam 5, two $\frac{3}{4}$ in.; beam 6, two $\frac{7}{8}$ in.; beam 7, two 1 in.; and beam 8, two $1\frac{1}{8}$ in. His concrete was proportioned one part cement, two parts sand, and four parts ballast, graded between $\frac{1}{4}$ in. and $\frac{1}{2}$ in. The age at testing was four months.

It would be reasonable to take for such a concrete a true ultimate strength of 1,600 pds./in^2, giving a working stress of 400 pds./in.^{2} For reasons previously given this would mean on the usual basis stresses of 2,000 and 500 pds./in.^{2} respectively. The yield point of commercial mild steel is often round about 40,000 pds./in^2 with an ultimate of 60,000 pds./in.^{2}. The tensile stress realised in reinforced concrete beams is somewhere about halfway between the yield point and the ultimate. Adopting a factor of safety of 4 and a working stress in the concrete of 400 pds./in.^{2} throughout we find by the author's new theory that Dr. Faber's tests would require a working stress of 11,000 pds./in.^{2} in the beams with 1 in. and $1\frac{1}{8}$ in. bars, 12,600 pds./in.^{2} for the $\frac{7}{8}$ in. bars, 13,000 pds./in.^{2} for the $\frac{3}{4}$ in. bars and 13,800 pds./in.^{2} in the third beam having $\frac{1}{2}$ in. bars. The variation is 25 per cent. These results work very nicely. The second beam with the single $\frac{1}{2}$ in. bar and the fourth beam with the $\frac{5}{8}$ in. bars are the exceptions; on the new theory they would afford stresses of about 500 pds./in.^{2} in the concrete and 16,000 pds./in.^{2} in the steel.

For isolated beams these results are quite reasonable. The author considers, therefore, that Dr. Faber's tests rather go to substantiate the new theory.

As regards Dr. Faber's other experiments on doubly reinforced beams the only criticism the author desires to make is that no lateral anchors were employed to prevent the compression steel buckling sideways, and that therefore the full strength could not be developed in the compression boom.

In the tests conducted at Manchester for the Concrete Institute's Sub-Committee on High Tensile Steels, duplicate beams were made, and the results were very consistent. They were tested at the age of 10½ months Being of good quality concrete they are free, at any rate, from Dr. Faber's criticism of Prof. Talbot's beams. Assuming an ultimate stress of 2,600 pds./in.^{2} and elliptical stress distribution the author's new theory gives the stress realised in the steel in the following table:

CONCRETE INSTITUTE'S TESTS.

(Breadth of slabs 21 inches. Effective depth 6·06 to 6·19 inches.)

Mark.	Kind of Steel.	Percentage of Steel.	Resistance Moment.	Stress Calculated.
A	Mild	·70	272,200	54,600
B	do.	·71	278,700	53,600
C	do.	·58	235,200	54,000
D	do.	·58	229,400	54,200
G	Indented	·46	251,500	75,000
H	do.	·46	247,900	74,800
J	Spiral Bond	·46	249,500	74,200
K	do.	·46	260,600	78,000
L	Drawn Wire	·46	309,200	94,000
M	do.	·46	297,200	91,000

The tests on the steel used showed the following approximate values (in pds./in.^{2}) for yield point and ultimate strength respectively:—

Mild steel: 46,000 y.p., 60,000 u.; indented bars, 64,300 y.p., 95,500 u.; spiral bond bars, 64,300 y.p., 81,400 u.; bright drawn wire, 90,000 y.p., 95,000 u.

It will be seen that the stresses realised in the steel in all cases were approximately halfway between the yield points and the ultimate strengths. The actual factor of

safety realised by comparison with the working strength computed by the ordinary theory was 3.6 in the case of the mild steel reinforcement. Attention is directed to the beams reinforced with drawn wire where the same percentage was employed as for other special steels, the only difference being in tensile strength. The stronger material gave the greater moment of resistance just as in Prof. Talbot's tests, for which the ordinary theory does not account.

Dr. Faber's remarks in this discussion might lead readers to believe that he did not agree with the author in having regard to ultimate resistance in practical design. However, in the issue of " Concrete " for May, 1916, he says: " It is really the factor of safety on ultimate loads which is of most immediate practical value. An engineer is more concerned with what load his beam will carry than at what load the proportionality of load and deflection ceases. Indeed, even if there were no experimental difficulties in the elastic limit method, it is doubtful whether it is always what is wanted. Suppose, for example, beam ' A ' will carry 2 tons without exceeding the elastic limit of any of its constituent parts, and will not fail before 8 tons, and beam ' B ' will carry 2 tons within elastic limits, and will fail at 3 tons, most engineers would be willing to load ' A' up to twice ' B.' "

Dr. Faber has misunderstood the remarks in the paper about bending moments in pillars in monolithic construction. The author was contrasting two tendencies that were in conflict. If the wording is read carefully it will be found that there is no confusion in the statement.

Dr. Faber makes the assertion that the paper sets out to prove that the usual method of designing slabs is all wrong. That was not the purpose of the paper, and seeing that the new theory gives similar results to the ordinary theory for moderate percentages of steel, and that most designers have not used large percentages, the paper agrees that the usual method of *designing* is right, though it contends that the *theory* is wrong.

Dr. Faber's remarks in regard to the table given on page 160 are also ineffective, because the table itself shows that an increase in thickness will reduce the percentage required by the ordinary theory. This, however, was not the purpose of the comparison.

In regard to Mr. Marsh's remarks it seems to the author most unscientific to base engineering calculations

on hypotheses which are " to some extent untrue." Mr. Marsh remarks that it seemed to him that if the author's new methods of calculation were generally adopted some very dangerous structures would be erected. Mr. Pimm makes a similar observation. An entirely adequate answer is the contribution by Mr. De Vesian, which was wholly unexpected by the author, and delights him very much.

The Hennebique formulæ were made to fit results of experiments and experience; they have been used for a very large amount of most successful work, and M. Hennebique's organisation used his formulæ to design many more reinforced concrete structures that have been done by any other designer. Mr. De Vesian has pointed out for many years that the ordinary theory must be wrong, and the author is very glad that in Mr. De Vesian's opinion his new proposals are based on sounder principles. The method of computing columns is the same as that of M. Hennebique and Mr. De Vesian shows that as regards rectangular beams the method which the author has tried to derive in a rational manner gives results in practical agreement with M. Hennebique's method of designing slabs. That is the greatest support that could be given as regards the practical utility of the new theory.

The author is also glad to have so much support from Mr. Edward Godfrey, who is one of the foremost American authorities on the subject, and a very original thinker.

Both Captain Lee and Dr. Faber criticise figure 28. It was thought that it would have been evident from the diagram that the ends of the construction shown would have been flexible and that the steel rod was not in contact with the concrete (being in a channel) so that the action would be that of a simple frame comparable with the portion of the truss shown in Fig. 22. Perhaps the adjoining illustration (Fig. 28a) of a similar imaginary construction will be clearer. When loaded the tension member must lengthen, and the compression member shorten, so that the ends would slide on the supports. It is hoped that this diagram will make it clear that a fundamental basis of the author's theory is the fact that the adhesion between the concrete and steel is practically non-existent in the later stages of loading about the middle of a so-called reinforced concrete beam, the pull in the steel being developed through the anchorage at the ends of the bars. Numerous tests have proved this, as also

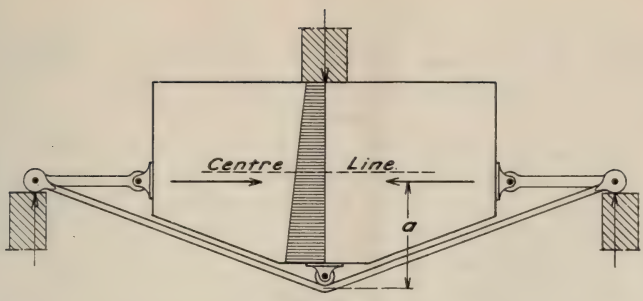


Fig. 28a.

the frame action that necessarily results. As references, see the Illinois University Bulletin, No. 71 (1913), on "Tests on Bond between Concrete and Steel," by Mr. Duff A. Abrams, and a still earlier paper by Mr. Charles Scott Fry, in the Proceedings of the American Society of Civil Engineers for October, 1910.

FOURTH INTERNATIONAL ROAD CONGRESS.

The Fourth International Road Congress will be held in May, 1923, at Seville, on the invitation of the Spanish Government.

The British Government will be officially represented by Sir Henry Maybury, K.C.M.G., C.B.

The Concrete Institute has been asked to appoint a representative on the Committee of the Congress, and the Council would be very pleased to learn the name of any member willing to serve in that capacity. Correspondence on the subject should be addressed to the Secretary, The Concrete Institute, Denison House, 296, Vauxhall Bridge Road, S.W.1.

LONDON BRIDGE ARCH.

Members of the Concrete Institute are invited to attend a public meeting to be held on Friday, July 7, 1922, at the Carpenter's Hall, Throgmorton Avenue, E.C., at 5.30 p.m., to protest against the proposed demolition of the ancient arch recently discovered under London Bridge.

The chair will be taken by His Grace the Archbishop of Canterbury.

A lantern lecture on the history of London Bridge will be delivered by Mr. W. D. Caroe, M.A., F.S.A., F.R.I.B.A., and an address on the history of Bridge House Estates Trust will be given by Miss E. Jeffries Davis, University Reader in the History and Records of London at London University.

It is hoped that the arch of the Bridge may be open for inspection by those interested between the hours of 4 p.m. and 5 p.m. on the same date.

NEW BUILDINGS, ETC., IN LONDON IN WHICH CONCRETE IS BEING USED.

BETHNAL GREEN. Five storey Warehouse Building for Messrs. Pearks Dairies Ltd. Reinforced Concrete construction throughout. Architect, J. H. Storrar, Esq. Engineers and Contractors, D. G. Somerville & Co., Ltd., 120-2, Victoria Street, Westminster, S.W. 1.

SHAD THAMES, S.E. Extension of Wharf for Messrs. Butler's Wharf Ltd. Reinforced Concrete Platform on piling supporting mass concrete quay wall. Sheet pile facing below platform. Engineers, Messrs. D. & C. Stevenson, Edinburgh. Contractors, Messrs. D. G. Somerville & Co., Ltd., 120-2, Victoria Street, Westminster, S.W.

Messrs. CHARRINGTON'S BREWERY, MILE END, E. Reinforced concrete water cooling for Condenser House. Builders, Messrs. F. & H. F. Higgs.

THE BLIND EMPLOYMENT FACTORY, WATERLOO ROAD, S.E. Steel framed construction with filler joists and concrete floor. Architects, W. H. Woodroffe & Son, 5, Bedford Row, W.C.

THE NORTH KENT BREWERY, PLUMSTEAD, S.E. New Steel Framed copper house, constructed. Also size of existing foundations increased and existing cast iron columns strengthened by encasing with steel reinforcing rods and concrete. Builders, Thomas & Edge, Woolwich, S.E.

Messrs. FREEMAN'S PREMISES, 101-103, LAVENDER HILL, BATTERSEA, S.W. Existing Building reconstructed and additional floors added.

The new floors reinforced with expanded metal are supported on 18″ × 6″—55 lbs. New Standard Section beams and steel stanchions.

Surveyors, Edwin Evans and Sons, Lavender Hill, S.W. 11. W. Hammond, Builder.

EXTENSION TO Messrs. SPALDING'S FACTORY, DEODAR ROAD, PUTNEY, S.W. Steel framed one storey North Lighted workshops constructed under the 1909 Act to effect economy. All the external walls which are of 9″ thickness are carried at Ground Floor level upon steel girders supported upon concrete piers about 30 feet apart. Architect, H. G. Leslie, Esq., A.R.I.B.A., F.S.I., Dartmouth Street, Westminster, S.W. 1.

Engineers for all the above : Messrs. Young & Co., 6, Queen Anne's Gate, S.W. 1.

THE CONCRETE INSTITUTE
PUBLICATIONS.

"Transactions and Notes."—

Vol. IV., Pts. 3 and 4	(Each)	2s. 9d.
,, V., Pt. 1		5s. 6d.
Pt. 2		2s. 9d.
Pts. 3 to 6 (inclusive)		11s. 0d.
Vols. VI. (Pts. 1 and 2), VII. and VIII.		5s. 3d.
,, IX.		5s. 9d.

"Standard Notation for Engineering Formulæ" ... ... 6s. 3d.

In this Report the Science Committee provide a system of standard symbols in order that time and consequently money may be saved in writing down the calculations which all classes of engineers are making daily in following their professions. The idea is to provide means whereby mathematical equations can be typed more easily, and printed more cheaply, and also to lighten the student's task by providing a set of self-explanatory symbols which will become imprinted upon the memory almost automatically, and be retained there without effort.

"Report of Joint Committee on Loads on Highway Bridges" 2s. 8d.

Effect of speed increasing virtual load; Table of standard loading for bridges of three classes; diagram of typical loads; tractive efforts on various road surfaces; notes on heaviest loads occurring in different districts; bending moment diagrams for standard loading; shearing forces; calculated example; effect of width of tyre and diameter of wheel; maximum axle loads allowed in various districts: Acts and Orders relating to motor traffic; scale diagrams of heavy vehicles with Table of loads and dimensions; determination of bending moments and shearing forces for beams subjected to moving loads.

"Recommendations to Inspectors, Clerks-of-Works and Foremen concerning the execution of Reinforced Concrete Works" ... 6½d.

"Standard method of measurement of Reinforced Concrete in Building construction," as adopted by the Concrete Institute and the Quantity Surveyors' Association ... 1s. 1d.

This pamphlet contains a full description of the order in which the work should be taken, the items which should be measured and grouped separately, and the mode of measurement in each case for *in situ* and *pre-cast* work.

"A Standard Specification for Reinforced Concrete Work" 1s. 7d.

Reports to the Scientific and Industrial Research Department on the Research Work on Concrete carried out under the direction of the Concrete Institute (1917 1919) ... ... 1s. 7d.

Post Free (Inland).

*PROCEEDINGS.

The 111th Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W. 1, on Thursday, 27th April, 1922, at 7.30 p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous meeting as published in the *Journal* were taken as read and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association :—

Members.

CLARKE, PERCIVALE JOHN, F.S. Arc., 34, Castle Street, Liverpool; FRENCH, HERBERT ALFRED LEWIS, Karachi Port Trust, Karachi, India; GIDLEY, ALLEN, A.M.Inst.C.E., P.W.D., Kuala Lumpur, F.M.S.; MCGILVRAY, JOHN ALEXANDER, A.M.I.E. (Scot.), 7, King Edward Road, Jordanhill, Glasgow; MARTIN, GEOFFREY, F.C.S., &c. Rosherville Court, Burch Road, Gravesend; MURRAY, JOHN, F.R.I.B.A., F.S.Arc., &c., 11, Suffolk Street, Pall Mall, S.W.1.; ORMOND, ARNOLD, 142a, St. George's Road, Bolton; SHORT, ANDREW ARMSTRONG, Ty Melyn, Aberthaw, Glam.; REYNOLDS, TREFER JENKINS, B.Sc., A.M.Inst.C.E., L.C.C. School of Building, Clapham, S.W.; STICKLAND, HAROLD MEERING, A.M.I.Mech.E., A.P.C.M., 8, Lloyd's Avenue, E.C.; WATERHOUSE, PAUL, M.A., F.S.A., P.R.I.B.A., Staple Inn Buildings, High Holborn, W.C.1.

STUDENT.

MCBRIDE, WILLIAM, 13, Stainforth Road, Newbury Park, Ilford.

A paper was then read by Mr. E. F. SARGEANT, A.M.Inst. C.E., M.I.Mech.E. (Member), on (a) "The Preparation of Concrete Aggregates," and (b) "Moving Forms."

A hearty vote of thanks having been accorded to the lecturer, at the conclusion of the discussion which followed, the proceedings terminated.

COUNCIL.

Mr. Paul Waterhouse, M.A., F.S.A., President of the Royal Institute of British Architects, has been co-opted a member of the Council of the Concrete Institute.

HONORARY MEMBER.

The Council have unanimously elected the Rev. H. G. ROSEDALE, D.D., F.S.A., an Honorary Member of The Concrete Institute.

* This Meeting was preceded by the Annual General Meeting, of which a report appeared in the April number of the *Journal*.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *JULY, 1922.* **No. 7.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

Papers for Session 1922-23.

Members desirous of reading papers before the Institute during forthcoming Sessions are invited to communicate with the Secretary of the Institute, at 296, Vauxhall Bridge Road, S.W.1. The title of the paper should be stated, a summary of the subject matter should be enclosed, and a suitable date should be intimated.

Papers should not exceed 10,000 words in length for printing, and are subject to approval by the Literature Standing Committee.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s.	0d.
		Remainder ...	£2	2s.	0d.
ASSOCIATES	,, ...	... ...	£2	2s.	0d.
ASSOCIATE-MEMBERS	,, ...	... ...	£1	11s.	6d.
GRADUATES, STUDENTS	,, ...	... ...		10s.	6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

FLOATING DOCKS.

By

*S. F. STAPLES, M.Inst.C.E., M.I.N.A.

Like ships, floating docks may be classified historically according to the materials from which they are constructed, i.e., timber, iron, steel, and possibly reinforced concrete. The earliest floating dock of which any history has come down to us was, like the ships of that date, constructed of timber, and in point of fact this particular dock commenced its career as a ship. At the time to which I refer, that of Peter the Great of Russia or Charles II. of England, the captain of a British merchantman in the Baltic found it necessary to do some repairs to the bottom of his ship. Had he been in home waters, he would have beached the ship at high tide, and by means of tackles attached to her mastheads have careened her or pulled her down to such an angle that any of her under-water structure would be exposed at low water, and thus have effected the necessary repairs. In the tideless waters of the Baltic such a procedure was impracticable, and still the repairs had to be carried out. In the port in which these occurrences took place, there happened to be an old hulk much larger than the vessel to be repaired, and this the British captain bought and gutted. He then fitted a gate at the after end, cutting away her stern proper to enable this to be done, and filled in ballast so that the hulk rested on the bottom with her upper works above the surface of the sea. Warping his own ship inside the hulk, he shut the stern gate and pumped the water out. His ship was then inside a primitive graving dock, which, however, owing to the water being pumped out, was afloat. Thus it was that the first idea of a floating dock was evolved. The name of the hulk was the "Camel," and a lifting pontoon to this day is known as a camel. I have a print of a floating dock on this principle dated 1829, which appears to imply that not much progress was made in the century and a half following the original evolution of the floating dock idea. In fact, as far as one can trace, nothing further was done until it occurred to some engineer, whose name has not come down to pos-

* Paper read before The Concrete Institute on 23rd March, 1922.

terity, to build a dock with the ends open, and to get the required buoyancy by making the platform upon which the vessel rested hollow. He was met at once with the problem of how to obtain stability for his dock, as if a structure be totally submerged it has no stability unless the centre of gravity happens to be below the centre of buoyancy—a state of affairs which cannot exist when a ship is being lifted on a submerged pontoon. In early docks there were two rows of stability boxes, one on either side of the pontoon, which were attached to the pontoon by screws. In the course of lifting a ship by pumping water out of the pontoon, the boxes were lifted too, and when they were nearly out of the water the screws were turned, so that they were lowered to their normal water line, and the pontoon pumped up again. Thus, by successive stages, the vessel was lifted. This was a tedious proceeding, and the next real improvement was to make the stability boxes sufficiently deep that if they were attached to the pontoon they had freeboard when the whole structure was lowered to receive a ship.

These timber docks had, of course, a considerable displacement of their own, which had to be neutralised either by stone ballast inside or by pumping water into raised chambers in the side walls, which ballast might be run out and thus help to lift the ship.

The era of the timber dock appears to have come to an end about the year 1860, when Rennie and others commenced to use iron for the purpose. With the advent of iron came also the necessity for self-docking. It was an easy matter to patch and repair a timber dock, and the bottom timbers which were always in the water appeared to last for ever; but in the case of an iron dock it is necessary to coat the bottom from time to time if the life of the structure is to be prolonged. With attention, the life of an iron dock appears to be about 50 to 60 years in sea water, and longer in fresh or brackish.

It is not always practicable to lift a dock in a larger one for the purpose of painting or repairs, and various means have been devised from time to time to enable docks to dock themselves. It will be convenient to discuss these methods after I have dealt with steel docks, as these latter are self-docked similarly to the iron ones.

The year 1893 saw the first steel dock with which I am acquainted. This was a dock which was constructed by Smith's Dock Company, Ltd., of North Shields, for themselves. A somewhat interesting occurrence took place in

connection with the launch of this dock. It was a sideways launch, and on the day of the launch the tide did not rise to within 3 ft. of the proper height, but it was decided to launch the dock all the same, and it was dropped into the water quite safely.

When it was found that steel deteriorates in sea water more rapidly than iron, self-docking became more than ever necessary, and many methods of doing this were evolved. In fact, the classification of docks is very largely a matter of self-docking. To use biological nomenclature for a moment, the genus to which a dock belongs may be determined by the number of side walls it possesses, while the species is mostly settled by its means of self-docking (except in the case of single-sided docks).

As has been mentioned previously, the earliest floating docks were double-sided, and to begin with were made in one piece only, being in consequence not self-docking. This, the simplest of all types, is called the Box dock.

Floating docks are almost invariably made approximately rectangular in section, but in 1868 a large dock was built for the naval arsenal at Bermuda, whose cross section was more nearly semi-circular. This was done in order that the under-water portions of the structure might be got at by pumping water into the compartments of one side, and thus careening the dock. This operation, I am informed, was once done, though what happened to the boilers when they were heeled over to the angle required to get at the centre of the bottom of the dock I am at a loss to imagine.

In the year 1902, this dock was superseded by a modern steel one. The system of self-docking which was employed in this case is that known as the " Havana " type. In this type the side walls are continuous from end to end, and are carried almost down to the bottom of the dock. The pontoon or lifting portion proper is entirely separate from the walls to which it is attached by a series of fish-plates bolted on to brackets riveted to the walls and pontoon. There are two series of these brackets, one higher up than the other, on the faces of the walls. By disconnecting these fish-plate joints, the pontoon can be completely separated from the walls; but as these latter would not be stable alone, the pontoon is divided into three separate sections, and the process of self-docking is carried out thus, taking the centre section as an example:—As much water as possible is pumped out of the dock, and the joints attaching the centre section to the walls are unmade. The dock is then lowered

by the admission of water until the upper joint brackets on the walls are opposite those of the loosely floating section of the pontoon, and the joints are then remade in this position. Water is now pumped out of the dock, and the section of the pontoon is thus lifted clear of the water.

When the side walls are to be docked, water is admitted to one side of the empty dock, and the opposite wall lifted thus clear above the surface of the water.

In order to deal connectedly with the Bermuda dock, I have been led away from the Box dock to which I would like to return for a short while, as in the year 1912 two Box docks of very large size were constructed for the British Admiralty. These are sister docks, and have a lifting power of 32,000 tons, and were designed to take any vessel in our navy or auxiliary navy. In designing these, my firm took the heaviest battleship, the largest armoured cruiser existing, and the "Mauretania," as types of vessels to be dealt with. One of these docks was towed to Invergordon and the other to the Tyne during the war. I do not think I am giving away any official secret if I inform you that during the first year the dock was at Invergordon 100 ships were lifted on her. Consider for a moment the amount of coal, time and risk that were saved by the presence of the dock where it was required, in place of having to send these 100 ships to the Clyde, Tyne or Portsmouth to be docked. This, be it remembered, represents only one year of the dock's career.

One of the great arguments in favour of floating docks for naval purposes is that of mobility, and this is very amply borne out by these two docks, whose original stations were on the Medway and at Portsmouth. Had it been necessary to work them at their home ports, as would have been the case had they been graving docks, a considerable proportion of our destroyer fleet would have had to be detailed for the sole purpose of escorting battleships and cruisers to and from the docks, to say nothing of the extra time the capital ships would have been unavailable for their proper duties

These docks were made non-self-docking for special reasons insisted upon by the Admiralty.

One very simple form of self-docking dock is exemplified by the Sectional Pontoon or Rennie dock, so named after its inventor. In this case the side walls are continuous, but the pontoon is cut up into sections whose length is less than the clear inside width of the dock. The walls rest upon the pontoons, and are attached to the same by bolted joints.

When self-docking is necessary, one or more pontoons are unbolted, and brought out from the rest of the dock, and turned round so that they can pass between the walls of the remainder of the dock, which in the meantime has been lowered to receive them. When they have been towed into position, the dock is pumped up, and the pontoons are lifted high and dry as would be a ship. The whole of the sections of the pontoon are thus dealt with successively.

One advantage of this type of dock is that the different sections of the pontoon can be constructed simultaneously in different parts of a shipyard, and launched independently. When afloat, the sections are moored in line, and the side walls are then built on top of them. This system also allows a dock to be built elsewhere, and the pontoons towed through a lock or other narrow entrance to a basin, into which the dock as a whole could not enter.

The drawback to this type of dock lies in the fact that owing to the walls being situated entirely above the pontoons, they are somewhat shallow as girders, this type of dock being entirely dependent upon the walls for its longitudinal strength. Notwithstanding this defect, they have been built to take the very largest ships. When war broke out there was a dock building at Reval with a lifting power of 40,000 tons, and during the war another was completed at Nicolaieff with a total lifting power of 39,000 tons, both from the designs of my firm.

Another self-docking type is the Bolted Sectional. This is like a solid Box dock with pointed ends to the pontoon beyond the ends of the walls. The whole dock is divided into three pieces by transverse bolted or riveted joints running round its entire profile. Owing to the joints going right down to the bottom of the pontoon, this is as strong as a Box dock, and still is self-docking. This operation is carried out by taking the joints apart and using any two sections to lift the third.

Most methods of self-docking are in reality variants of one of those I have described, and it would only weary you to illustrate further types of double-sided docks.

Single sided docks were the invention of the late John Standfield, and were brought into being as the only method practicable of docking the circular ironclads built for the Russian Navy from the designs of Admiral Popoff about the year 1875. These circular ironclads had a diameter of 120 ft., and consequently could enter no graving dock. Standfield evolved the idea of making the dock one-sided,

with the pontoon in the form of fingers instead of being solid from end to end as had up till then been the custom. Some of these fingers were loose and could be attached to the ends of other fingers, thus doubling the width of the pontoon or lifting portion of the dock. In connection with the dock there was a grid erected along the water front, the spacing of which was the same as the spacing between the fingers of the dock pontoon. By interlacing the grid and pontoon, the ship was brought over the grid, and the dock being then lowered the ship rested upon the grid, the dock thus being available for other ships. This type is the well-known Depositing dock. Stability is obtained by means of a floating outrigger attached to the wall of the dock by a series of parallel hinged booms.

The self-docking of these docks is a particularly simple operation. The wall has a joint at the centre of its length; and if the dock be divided into halves by unbolting this joint, one section can readily be docked, so to speak, in the lap of the other.

The Depositing dock is only suitable for ports where there is little or no rise and fall of tide; otherwise the deposited ship would be flooded at high water spring tides, or if high enough out of water for this, would only be able to be deposited and lifted off again at unusually high tides.

For use in ports where there is a tide, another form of single-sided dock was evolved by my firm, known as the Off-shore. In this case the pontoon is not made in the form of fingers, but, with the exception of the joint in the middle, is continuous from end to end. It has booms, as in the case of the Depositing dock, but no floating outrigger, strongly braced columns erected on piles taking its place. Gear is fitted on the top of these columns to indicate whether in the course of lifting or lowering a ship the tendency is to tip outwards or inwards, so that the same may be corrected by the manipulation of the valves, and thus obviate any undue stresses upon booms or shore-work.

The Off-shore is an exceedingly useful and economical type of dock, economical both as regards first cost and as regards cost of working.

Taking the cost of construction first. The walls of a double-sided dock are primarily for the purpose of giving transverse stability to the structure while the pontoon is below water, and this consideration governs their width, and to a certain extent their length. In an Off-shore dock, their principal function is to prevent the pontoon from sinking

too rapidly, and to provide a working platform, as the transverse stability is taken care of by the booms and shore-work. The single wall is, therefore, usually much narrower than would be the case of a double-sider of the same size. Thus not only is one wall done away with, but the remaining wall is smaller. Against the saving of one entire wall is the cost of the shore columns and booms, which as a rule is a much smaller item.

As regards cost of working, it will be found on investigation that a large amount of the pumping horse-power is expended in ejecting water from the walls, which in no way helps to lift the ship. It is obvious, therefore, that if one wall is absent, one-half of this useless work is saved, and a further saving is effected by the remaining wall being of smaller width. To give an actual instance of this difference I have taken two docks of the same size and length and find that the actual saving in work done in lifting the same ship was almost exactly 100,000 foot-tons, or one-third of the total required to lift the ship.

These docks can be made very rapid, and in busy ports it is really astonishing to note the amount of work they can do. To give an instance of this, I might mention a small Off-shore Dock at Aberdeen, which in five years handled 1,589 vessels, an average of 318 vessels a year of 308 working days. The largest number of ships occupying the dock in the course of 12 hours was five, one being undocked, three successively docked for sighting, and a fifth docked for painting.

Another advantage of these one-sided docks lies in the fact that paint dries so much more quickly than it does in a graving dock. The late Mr. Eustace Smith, when managing director of Smith's Dock Company, Ltd., who at the time owned one graving and one floating dock, told me that there was quite a considerable difference in the time taken for paint to dry in his two docks, which to him was a very valuable asset.

Having now dealt with the principal types of floating docks, I will turn for a few moments to the internal construction of the same.

The more important stresses which come upon the material forming the hull of a floating dock are: (1) water pressure, (2) transverse bending moment due to the pressure of the keel of the ship upon the keel-blocks, (3) longitudinal bending moment due to the bearing length of the ship not extending as a rule over the full length of the pontoon.

(1) Taking the condition of affairs when the dock is floating with its deck just above the water and with a ship resting on the keel-blocks. The lifting power is the pressure of the water exerted evenly all over the bottom plating. Part of this pressure goes to support the weight of the dock, and the remainder has to neutralise the weight of the ship applied along the line of the keel-blocks. The evenly spread water pressure over the whole bottom, therefore, has to be concentrated into a single line running along the centre of the dock. The load on the skin plating is in the first instance taken from the skin to the frames, and from the frames to the transverse bulkheads which run right across the dock. These transverse bulkheads take the load off the frames, and transmit the same to the central bulkhead upon which the keel-blocks rest.

(2) The effect of the loads from the frames coming upon the transverse girders is to set up in them bending moments, the bottom skin plating forming the tension member and the pontoon deck the compression member.

(3) It being always impracticable to pump water from the dock exactly under the limits of bearing length of the keel of the ship, it follows that the water pressure on the skin of the dock beyond the ship bearing blocks has no weight to oppose it directly. Some portion of this finds its way to the loaded portion of the central longitudinal bulkhead along the bulkhead itself; but the major portion, when the load is considerable, has to travel outwards to the walls, and is by them transmitted to the ends of the transverse bulkheads, and so to the loaded portion of the central bulkhead.

When the dock, instead of supporting a ship above the surface, is lowered, some of the stresses in the skin plating will be reversed owing to the weight of the side walls being considerably greater than their net displacement, the bottom plating, treated as the lower flange of the transverse girders, being now in compression, and the pontoon deck in tension.

I have treated the stresses in some of the skin plating somewhat in detail to indicate the problems the engineer has to deal with when designing a floating dock in reinforced concrete.

During the war my firm (Clark & Standfield) was commissioned by the Admiralty to design a floating dock in reinforced concrete with a lifting power of 8,000 tons, but although full working drawings were got out the dock was not

built. This was principally due to the cost of preparing the basin in which the dock would have been built. If the launch of a dock be considered for a moment it will be seen that at first the dock is supported entirely upon the fixed launching ways. As the front part enters the water, it becomes more and more waterborne, until at last it lifts off the ways altogether. In the dock we are considering, this occurs when the leading portion of the pontoon drew 16 feet of water. In thus lifting, the dock pivots round the extreme end of the sliding ways underneath it, and subjects them to a very large local load, which in the case of a concrete dock would be very difficult to distribute so that no local damage would be done to its structure. This load in the case of the dock we worked out for the Admiralty was in the order of 3,450 tons, which would have to be taken up upon a very few feet of bearing surface. The ground under the fixed ways would have to be extensively piled in all probability to stand the load of 14 tons to the square foot which the above mentioned load would throw upon it, and as this piling would have to be carried out for a considerable distance under water the cost would be prohibitive. In these circumstances it appeared out of the question to contemplate a launch, and as no basin already constructed was available, a special one would have had to be built for the purpose for each dock required. The conclusion come to when the designs were completed was that although it is possible to build a dock of this size, it could never be a commercial proposition.

One of the great difficulties in designing floating docks in reinforced concrete lies in the great weight of material in the skin plating. The minimum thickness we could use towards the centre of the bottom plating of the pontoon was 5 ins. thinning down to 4 ins. at the outsides. This only gave ½ in. of cover outside. This cover is hardly enough for safety in ordinary circumstances, and when it is considered that a very large portion of it cannot be inspected after the dock is once afloat I should not care to recommend it. Moreover, the difficulty of getting about an acre of this thickness dead flat would be considerable. On the other hand, if more cover be added the weight goes up considerably, and the dock has to be of greater dimensions to carry the extra weight, thus inordinately adding to its cost.

Even by keeping the weights down as low as possible the total weight of the 8,000-ton concrete dock was 9,570 tons as compared with a steel dock of the same dimensions

on plan of 3,584 tons. This necessitated the concrete pontoon being made 16 ft. deep, as against 10 ft. 6 ins. for the steel one. In order to lift an 8,000 ton ship in 2½ hours in a steel dock 350 I.H.P. were required, whereas an additional 150 I.H.P. were required to lift the same ship in the same time on the concrete dock.

During the latter portion of the war I was Director of Reinforced Concrete Construction at the Admiralty, and had to look after 23 shipyards solely engaged in constructing reinforced concrete vessels. As a result of that experience I may say that it has been fairly conclusively demonstrated that reinforced concrete is not an ideal material from which to build a ship, and to my mind the same remark applies to a floating dock. It must be borne in mind, too, that joints in a dock would be very difficult to make, and consequently a concrete dock would not be self-docking. If, therefore, any of the skin of the dock under water cracked in the slightest degree, there would be no means of detecting this, and the reinforcement would slowly decay until brought to light by some disaster. When a floating dock is moored in a busy harbour there is a certain risk of collision, which in the case of a steel dock is no great matter, but with a concrete dock it would be exceedingly difficult to repair, partly owing to the nature of the material and also to the fact that the under-water portions cannot be brought above the surface.

For the above and other reasons it appears that a floating dock of the usual form should not be made of reinforced concrete, but there is another form in which it might be found advantageous to make use of this material which is worthy of your attention. If a reinforced concrete floating dock were designed with the longitudinal and transverse girders of normal strength, but with the skin plating only just strong enough to enable the same to float, and one end were closed in a manner similar to that in which the side-walls were constructed, this would form a complete lining to a graving dock. Where the ground was bad, the whole of the excavation could be done in the wet, and the floating lining towed into place and lowered so that it rested lightly on the bottom. Weak concrete could be forced down pipes led through the pontoon deck and bottom so as to ensure a firm and even seating for the floating dock, which would then be filled up solid with weak concrete. The deformation of the walls of a dock thus constructed should be very

small, so that the sill and side facings for the caisson to bear against would be built before the floating dock was lowered, and the whole of the cofferdam and pumping during construction would be done away with.

The author was connected with a somewhat similar scheme some years ago in connection with the new harbour of Kobe. Here the quay walls were constructed of hollow blocks of reinforced concrete about 120 ft. long, varying from 35 ft. to 41 ft. high, and having an average breadth of about 30 ft., the finished weight being in the neighbourhood of 6,000 tons. These were built on a grid erected on the edge of the sea, and the blocks were picked off the grids by a depositing dock, and subsequently lowered into deep water by the dock. Before filling they weighed about 2,000 tons, so that they could be then towed to their destination and sunk by the admission of water. This proved a very successful method of construction, and with modifications has been followed in other harbours.

In writing a paper on floating docks, it is almost inevitable that one becomes involved in a discussion of the relative merits of floating docks and graving docks, but in this case it would appear to be uncalled for, when, as was seen above, the same material starts its life as a floating dock and continues and ends it as a graving dock.

DISCUSSION.

Mr. H. J. Deane (Vice-President) proposed a vote of thanks to the author. He referred to an old type of dock which was in existence in London a good many years ago, and which was built in the Victoria Docks It was in the nature of a hydraulically operated dock, and the ships were floated on to the pontoons by means of hydraulic pressure. It necessitated the construction of a large power house, and a large consumption of water was necessary; the scheme was intended to be an economical and commercial scheme, but he believed it had failed. On the subject of floating docks or graving docks generally, there was a very large amount of discussion as to whether floating docks were the right thing, or whether graving docks were the right thing. He personally did not take any hard and fast view on the subject, because each particular type had its own particular uses. The small reinforced concrete dock had its own particular uses during the war, and, as the author had said, the utilisation of reinforced concrete floating docks of large capacity was out of all possibility of execution. Therefore, they must consider the circumstances under which floating docks were to be used before deciding upon their construction. A great argument in favour of floating docks was that

they could be transported, and from the point of view of the Admiralty they were most excellent tools. Taking a river such as the Thames, if they examined the contours of the bottom, the width, etc., he believed they would come to the conclusion that there were very few spots on the Thames where they could safely place a dock of this description without having to undertake a very large amount of dredging, and possibly a very large amount of excavation of the foreshores. There was one place where it might be possible to put such a dock, but, on the other hand, that place was situated in a position of the river where probably in the future there would be a large amount of commerce going on, and the dock would be in the way. With regard to wet docks, it was impossible to conceive the construction of one of these floating docks inside a wet dock, because it would not only mean the excavation which was necessary for the construction of the wet dock, but also the necessary quay walls which would be necessary to keep up the sides, and so on. The wet docks, therefore, must have, for the sake of convenience, their own graving docks within their own boundaries, so that ships could go into the docks to receive the necessary treatment. With regard to the old "Victory," there was very good reason why she was lying in a graving dock and not a floating dock; the utilisation of a floating dock would be uneconomical, and would sterilise the use of that dock. He was not quite in agreement with Mr. Staples, however, in regard to the "Victory." He understood that she was suffering from dry rot inside, and not from rot outside. Dry rot inside was a thing which might start anywhere in any wooden building, and required very prompt and careful treatment, otherwise it would spread hopelessly. It was due to a large measure to want of ventilation, and that was really what the "Victory" was suffering from, and not from the rot caused by the water. He did not think the author had referred to possible accidents in connection with floating docks. It was not in every graving dock that they had bilge blocks, and for the smaller ships they relied solely on the keel blocks. The ordinary method of taking a ship into a graving dock was to float it through the dock gates, centre it, and then gradually pump the water out until the ship was just about on the keel blocks; then they proceeded to put in the shores, which were gradually tightened up as the ship settled on her keel blocks. During the war the difficulty of getting sound and well seasoned timber was a very serious one, and they had to put

up with what they could get, and the result was that he had got some unsound wood on one occasion. There was a small vessel in the dock, and when the water had been pumped down to within 12 ft. of the bottom one of the shores carried away, with the result that the stresses on the remaining shores on that side of the vessel were so great that they also were carried away. He (Mr. Deane) had been sent for, and had found the ship lying on its side, and he would like to know what might have happened had the same thing occurred on a floating dock. When the dock was dried, and the vessel shored up to prevent collapse, it was found that she was still sitting on her bilge blocks. The nautical men —who were, of course, omniscient in these matters—suggested getting blocks and tackle on to the mast, filling the dock with water, and pulling her up. He, however, would not take the responsibility! The vessel had been put on very high keel blocks, which were placed about 8 ft. apart. They built up additional keel blocks, and jacked the vessel up until she took on a little more list. She was just on the point of over-balancing, but that was counteracted by putting in shores on the lower side. The top timbers were then taken from the original blocks, substituting for them some soft wood packings, 6 in. less in height. The jacks were gradually released, and the vessel began to roll back to her correct position. They proceeded until the keel was resting on the bottom timbers of the keel blocks, about 20 ins. above the dock bottom, and came to the conclusion that it would then be a simple matter to float her. What actually happened was that they let the water rise, gradually at first, and got the dock full of water before the tide was high. She was then taken to the wet dock, with a list of about 8 degrees, due to some of the ballast inside having shifted. The only damage done was that three or four rivets had been carried away, and they had to be plugged. He very much doubted whether that could be done in a floating dock. Again, with regard to floating docks, there was the question of the influence of the tides; it was obvious that these big docks must run a great deal more risk in a seaway than a graving dock. Therefore, he imagined that they always had to be placed in a position which was more or less sheltered. Nevertheless, he had a great admiration for them, and considered them to be the right things in the right places, provided they could get the depth of water without very much dredging, the necessary width, etc.

Mr. Leverton seconded the vote of thanks.

Mr. Yeatman, referring to the accident mentioned by Mr. Deane, said he thought the floating dock would be an advantage in such a case, because they could correct matters by pumping out one side and giving the dock a cant over. Also, he believed the ship could have been pulled over by the mast if there were plenty of water in the dock.

Mr. Deane said that the edge of the deck was below the water line, and that very materially altered the stability of the ship. Had that not been so, it would have been a simple matter to have raised the water, and she would have floated naturally.

Dr. Salmon mentioned an experience when something of the same kind as that mentioned by Mr. Deane had occurred. His firm had made a floating dock for Penarth, and it lifted about 4,000 tons. Owing to the test ship not having arrived, they tried to lift a 200-ton Russian schooner. The ship was pulled into the dock and centred on the keel blocks, and when she was about three-quarters of the way up it was noticed that she was slipping off the dock. Subsequently it was found that the ship had a 2 ft. bar keel, and it was found that she was not touching the bilge shores at all. The vessel had been pulled into position by mast head tackle. To keep her in position the dock master had put piles of timber in the middle of the long baulk of timber which formed the bilge shore, and the result was that the ship settled on the bilge shores, and the keel was resting on about a foot of timber fore and aft. However, the mast head tackle held the ship until she was properly shored up. Mr. Staples had not mentioned one type of floating dock, which was an idea of his own. The idea was to make a pontoon which was an open tray, and to moor it in a place where there was a fairly hard bottom and a certain amount of rise and fall of tide. When the tide went out the tray rested on the bottom; there was a valve in the bottom of the tray, and when the tide returned the tray filled with water and rested on the bottom of the sea. A ship was hauled on to the tray when the tide was in, and when it went out again the valve was closed; then when the tide came in for the second time the whole thing was raised. There was no pumping power involved, and this was a very convenient way of dealing with small craft; the idea could, of course, be developed more by introducing pumping machinery. He was glad Mr. Staples had emphasised the value of the off-shore dock, because people did not realise

what a good thing it was. For small ships he believed it was the best. As to the hydraulic lift designed by Edwin Clark, whereby the ship was lifted bodily out of the water by means of hydraulic pressure, put on to a pontoon which was pulled out, and the dock was ready to lift the next ship, it was a curious thing that during the war a great number of patents had been taken out in Germany for an exactly similar thing, but instead of using hydraulic pressures they had a small floating dock, and there was another bigger dock which lifted the smaller one bodily out of the water. That was a modern development of the hydraulic lift.

Mr. White said there was one thing which would have been of interest to the members, and that was that in some docks there was an air space running through the centre of the pontoon. He did not think that had been mentioned in the paper. It had occurred to him that if there were an air space in the pontoon, why should they not have water in the side walls, so that when they wanted to lift the dock they could pump the water out of the pontoon, leaving the water in the side walls, then open valves in the side walls and let the water run out by gravity. That would be saving pumping power to a certain extent.

Mr. Sinclair, speaking of side shores in a dock, said he took it that those shores were limited, inasmuch as the range was limited. He asked how Mr. Staples would get over the difficulty if he had a much narrower ship in the dry dock, so narrow that the side shores would not reach it. He presumed that portable shores would have to be used in their place. With regard to mooring floating docks, they took up a lot of room, and when they were out of water there was a large amount of the dock and the ship exposed to wind pressures; therefore, if such a dock were in an exposed position when a gale was up the action of the wind on the side of the dock, and the ship itself, was a very serious matter. He would like to know what arrangements were made for mooring, because it was a case of a vessel in a vessel, and the outside one had to be moored. Then, as to the area occupied, if they had to throw out anchors, or put mooring posts or bollards on either side, it would take a considerable area, and he asked for information on that point. With regard to the sectional self-docking docks, he asked for information as to how the joints between the sections were made, because there were no doubt heavy stresses set up in the side walls of the dock, especially when they

had a very heavy dock where the stresses were unevenly distributed over the whole. As to the one-sided dock used in connection with grid irons, he took it that their action was limited to particular types of vessel. It would be very convenient to pick up a flat-bottomed ship, and put it on the grid, because it would rest on the flat bottom. If they had a ship which ran away quickly to the keel it would be more difficult, and he would like to know how to get over that difficulty.

Mr. Marshall referred to the slide depicting a self-docking dock, in three sections, the ends of the two outer sections being tapered; when self-docking these two tapered ends were turned round and raised the centre section. He would like to know how the end parts were docked. As to wind pressures, about fifteen years ago he had seen a floating dock at Singapore. This dock had been towed across the Atlantic, through the Mediterranean, through the Suez Canal, and across the Indian Ocean. It must have met some storms, and, although it was not carrying another vessel, that alone was a good feat.

Mr. Staples, replying to the discussion, expressed his thanks for the appreciative words which had been uttered. He agreed with Mr. Deane that there was plenty of room for both types of dock, both floating and graving; neither were perfect for any and every situation. Although he designed floating docks, he had written to his clients in some cases and pointed out that their particular port was not suitable for floating docks, and recommended graving docks. As to Mr. Deane's question as to the giving way of the side shores, letting the ship over, in the ordinary way a floating dock did not have side shores. It had bilge shores, which were solid blocks of timber, with a steel base, so that there was no risk of a vessel falling from the bilge shores any more than from the keel blocks; accordingly, an accident of that sort could not happen. There were only two or three side shores on either side, and these were merely used for the purpose of centring the ship. Therefore, from that point of view they were much safer with a floating dock than with a graving dock. The idea of having a steel base to the bilge block was that the block was built up in the form of a wedge, and was attached by sliding clips on to the top of the steel base, and by means of a chain which went up to the top of the dock wall, when the ship was on the keel blocks the wedge was pulled in until it was tight against the side of the ship.

By that method they could be certain that they had the majority of the weight of the ship on the keel block. As the weight of the ship became greater the keel blocks compressed, and the ship was firmly resting on the bilge blocks on either side. The accident that Dr. Salmon had mentioned did not occur when really docking a ship, but when performing gymnastics. They did not expect a ship going into a modern dock to have a 2-ft. bar keel. Beyond that, he had never heard of any accident happening in a floating dock. If they had a ship with an enormous list, it was difficult to deal with in a graving dock, but with a floating dock one side could be lowered so that the dock would be at such an angle as would correspond to the list of the ship. That had been done constantly in war time, and they could do in ten minutes what had taken four days to do in the graving dock, referred to by the first speaker. With regard to mooring floating docks in sheltered position, he mentioned a number of cases in which they had been moored in the open sea, including a dock at Valparaiso, which was moored in the open Pacific. When the dock was sunk there was very little for the waves to get hold of. When the dock was sunk there was a big, flat pontoon, which had to be dragged up and down, and there was only the displacement of the side walls to drag that up and down. Consequently, there was a firm base for the ship to rest on, and there was no chance of the dock and the ship bumping.

Mr. Deane asked whether floating docks in exposed positions were sunk during a storm.

Mr. Staples replied that they were not, but they merely rode it out. Their limit was practically if the waves were more than 4 or 5 ft. high; if they were higher they would not attempt to dry-dock, unless in case of emergency. With regard to currents, a floating dock was better in a current than a graving dock, because the dock would always lie up and down the current, which was useful in a river. Frequently a graving dock had to be placed at right angles to the river, and at right angles to the current, and it was a much more difficult problem to get a ship in than when the current was straight up and down, and a tug could steam straight through. With regard to air spaces in the pontoons, a great many docks were made with air spaces, such as described by Mr. White. Also, they were made so that the water could be run out of the walls by gravity, instead of pumping, but when they counterbalanced the weight of the dock it meant that they had so much less weight to sink.

and it might take quite a long time to sink the last 4 or 5 ft. It meant extra water-tight decks, and extra ballast, and would add considerably to first cost. As to the length of side shores, they were made as long as they could be made, consistent with strength. From time to time, where a dock had to deal with a large variety of ships, they made a hinged nose on the side shores. They put a hole through the side shore a couple of feet from the end, and on either side of that a cambered channel, and clamped another piece of timber on, say, 10 ft. long. That could be hinged up in the air when dealing with a wide ship, or lowered for a narrow ship. That had proved very satisfactory. As to mooring, a floating dock did not necessarily take up a vast amount of room in a harbour, because when moored by chains, the chains were attached to the side of the dock, they were lowered through a ring, and then brought across underneath, so that the anchor did not come so very far out from the other side of the dock. Moreover, the chain naturally sloped down, and by the time it came to the wall it was perhaps 25 ft. below the bottom of the dock, and was no obstruction to navigation. They could also be moored by beams or between bollards, according to circumstances. With regard to the joints between the sections of a dock, where they were above water it was simple. They made the joints with a butt cover, and rivetted that on. It was only once in about 10 or 15 years that a dock had to be taken apart for self-docking, so that if the joints were bolted the bolts might become rusty, and it was just as simple to cut out a rivet as a bolt. Below the pontoon deck there were right angles rivetted to each end of the dock. The dock did not end in a bulkhead, but bulkheads formed the ends of the sections, and there was a joint chamber. These were rivetted at the joint chamber. Allowance was made for a rubber core, generally about 1-in. in diameter, which would make a water-tight joint for the head of water which the dock would take at, say, 5 or 6 ft. They made the rivet and joint on the top. They let a little water in each end of the dock, which caused the bottom part to nip in, and a man was sent down to bolt up the joints. The same thing happened when undocking. They could make the joints as strong as the plating of the dock, and there was no reason why a dock should break at the joints, any more than anywhere else. With regard to dealing with ships deposited on the grid, they had exactly the same system of bilge blocks there as on the dock. The ship was lowered until it rested on the keel blocks, and

then bilge blocks were built up on either side, and the ship was just as firmly supported as when on the pontoon deck of the dock. Mr. Marshall had enquired as to how the end sections of a three-piece dock were self-docked. The pointed end was turned into the square end of the middle section to self-dock the middle section. If they wanted to self-dock the end section, the pointed part of the other end section was put underneath the square end of the end section to be lifted. Its own point was then raised on to the pontoon deck of the middle section, and it could be lifted quite simply. With regard to the stresses which a dock would stand, it was sufficient to say that they were towed all over the world, and could be towed anywhere. It was just a case of calculating stresses in the same way that those stresses were calculated in the case of a ship.

PROCEEDINGS.

The 112th Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W. 1., on Thursday, 22nd June, 1922, at 6 p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the minutes of the previous Meeting as published in the *Journal* were taken as read and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association :—

Members.

AGA, BURFOR L., F.S.Arc., Lic.R.I.B.A., Morarbhoy Building, Bombay; AUSTEN, EDWARD MONTAGUE, Amee Villa, Gondivli Road, Bombay; *BEST, HALSTEAD, M.S.A., 46, Read's Avenue, Blackpool; EARLE, GEORGE FOSTER, M.C.S., Messrs. G. & T. Earle, Ltd., Wilmington, Hull; FORDHAM, ARTHUR A., A.M.Inst.C.E., Barnfield Villa, Hathersage; WILSON FREDERICK C., M.S.A., Garrison Engineer, Baluchistan, India.

*Transferred from Associate-Membership.

Associate-Members.

CONNELL, WILLIAM R. F., 82b, Bedford Road, Clapham, S.W. 4.; GREENFIELD, THOMAS, M.S.A., Easebourne, Midhurst.

Graduates. (Elected by the Council.)

BLYTHE, PHILIP WAKEFIELD, 30, Olive Mount, Tranmere, Birkenhead; GOMERSALL, J., 51, Princess Street, Salford; HABERSHON, MATTHEW EDWARD, Ferham Villa, Rotherham; SELLMAN, THOMAS D., 111, Alexandra Road, Birmingham.

Resignations.

The Secretary reported that the Council had accepted the resignations of the following Members :—ALFRED HANKINSON, London, elected 1911; D. C. RATTRAY, Southport, elected 1909.

Deaths.

Sympathetic reference was made to the loss sustained by the Institute through the death of the following members :—P. W. TOLHURST, elected 1909; ROBERT WALKER, elected 1909.

The proceedings then terminated.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *AUGUST, 1922.* **No. 8.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s.	0d.
		Remainder ...	£2	2s.	0d.
ASSOCIATES	,, ...		£2	2s.	0d.
ASSOCIATE-MEMBERS	,, ...		£1	11s.	6d.
GRADUATES, STUDENTS	,, ...			10s.	6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

NOTICE.

In view of important resolutions which the Council intends placing before the Members very shortly, it is hoped that all Members whose subscriptions are not fully paid up will make a point of setting this matter in order, so that every Member may be in a position and qualified to vote. Particulars and statements of accounts may be obtained on application to the Secretary. (Telephone: Victoria 2112).

* THE PREPARATION OF CONCRETE AGGREGATES.

By E. F. SARGEANT, Assoc.M.Inst.C.E., M.Inst.M.E., M.C.I.

A cubic yard of reinforced concrete consists of 30 to 33 cwts. of aggregate, 4½ to 5 cwt. of cement and 1½ to 2 cwt. of steel; so that the aggregate weighs six to seven times as much as the cement, and 15 to 20 times as much as the steel; therefore although a suitable aggregate may be bought at a fixed price delivered on the ground yet the labour cost of dealing with that aggregate may become a very serious item. This labour cost becomes a question for consideration in the lay-out of the whole job, but the plant for handling the aggregate may be modelled on the lines of the plant for preparing the aggregate, which the Author proposes to deal with in this paper, although he will be pleased to deal with the former subject in the discussion, if found of sufficient interest.

There are two intrinsically different methods of preparing aggregate for concrete, according to whether the material employed consists of naturally formed gravels or of rock which has to be reduced to the necessary fineness. In the case of gravels the chief work to be done is washing and separation into large and small aggregates accompanied by, perhaps, a little crushing to reduce the larger lumps, and perhaps to increase the quantity of sand; therefore a gravel preparing plant consists essentially of screens with a plentiful supply of water for effecting the washing and separation, with the addition as an auxiliary to the plant, of a crusher of sufficient size to take the largest stone likely to be met with.

In rock crushing, on the contrary, the crushing plant forms the most important part, the screens being subsidiary; all the material as it is delivered from the wagons passes into the crusher, and from thence into rolls, and the crushing and screening can usually be done without the use of any water.

We can, therefore, divide aggregate producing plant into two broad types, namely, those in which the material is rock and those in which the material is gravel; the former

* Paper read before the Concrete Institute on 27th April, 1922. It is hoped to reproduce the diagrams in a subsequent issue.

have large and powerful crushers through which the material passes on its way to the screens; the latter have the material delivered direct into the screens in which it is washed and separated, the rejects or oversizes being led into the crushing plant, if they occur in large enough quantities to justify this expense.

The first diagram shows a general arrangement for producing crushed limestone 2½ ins. downwards as installed by Messrs. Edgar Allen & Sons, Ltd., of Sheffield, to whom the Author is indebted for the wall diagram. The stone is brought from the quarries by truck and discharged into the large jaw crusher shown on the right of the diagram. The above sounds as simple as the world-famed Mrs. Glass's injunction to " first catch your hare "; the technique of quarrying is, of course, a study by itself, but in designing the crushing unit a size of crusher must be chosen which will deal with the rock as brought from the quarries usually rock which is hard enough for concrete, has to be loosened in the quarries by blasting, the larger lumps broken up by smaller shots, and the debris picked up by large grabs and loaded into wagons to come to the crusher plant. The Author is inclined to think that the American plan of using a breaker ball, attached to the crane, for breaking up the larger lumps might be successfully introduced into English quarries, as the cost of blasting and reducing the lumps to a size suitable for the jaw crushers is about 1s. 8d. per ton for hard rock limestone. The product from the jaw crushers can then be taken to a smaller set of jaw crushers, and then passed through rolls; the diameter of these rolls depends upon the size of the material delivered by the crushers, big material being, of course, apt to fly out of smooth rolls. To overcome this the rolls are sometimes toothed or fluted, which gives them a better grip of the material. From these rolls the material is elevated by bucket elevators, as shown in the figure to overhead screens from whence the product is taken to various storage bins whence it can be run off by appropriate chutes to wagons for distribution. The screens shown on the illustration are of the telescopic variety; that is, they are placed one within the other, the screen with the largest holes being inside. The general features of screens will be dealt with later on. The plant shown in the figure is of very simple construction, and runs with practically no attention, except that of feeding the stuff into the crushers and removing the finished products from the various chutes; it will produce about 25 tons per hour of crushed limestone screened from

2½ in. downwards and the power required is about 100 h.p. As a rule most rock breaking plants that the writer has seen are fairly well designed to economise labour, but in gravel screening and washing plants the case seems to be quite otherwise. Even in rock crushing and screening plants, however, too little attention seems to be paid to the capacity of the various bins; they are usually too small to take any but a few hours' work of the plant, and when they are full the stuff either has to be wheeled away and dumped, or the plant has to be stopped. Now the cost of a concrete storage bin for stone is only about £2 15s. to £3 10s. per cubic yard of storage capacity, so that it pays to instal large bins, as the cost of dumping and reclaiming crushed stone is fairly high unless steam grabs are employed. The Author may mention that he has constructed concrete storage bins for grain in the bulk at 9s. per cubic yard of storage capacity, but this price could not be approached for the smaller bins required for stone, and did not include foundations or roof.

The second diagram shows side and end elevation of a gravel screening plant for concrete. It consists of a couple of storage bins, one for the sand and one for the coarse aggregate, the latter being, of course, twice the length of the former; these bins are preferably mounted well above the ground on posts, so as to allow the finished material to discharge down inclined chutes to wagons, belt conveyors, or the concrete mixer, as the case may be. The cost of mounting the bins up in this way is very little, as is also the cost of elevating the raw material to the hopper. These bins must be well braced sideways and longitudinally to resist wind action, and also the vibratory action of the crusher, the nature of this bracing being chosen to suit the locality, being either internal cross bracings, or external guy ropes or raking shores.

At the top of the whole structure is a capacious water tank; this is not often installed, but it is of great utility if it can be done without excessive cost. It is of utility because the water supply for washing is usually from a pump (not shown in the figure), and as this water supply has to be cut off when not required, it is preferable to do this by a cock near the jet rather than by starting and stopping the pump on each occasion; if the cock is shut off while the pump is running, and there is no overflow accidents would happen, and therefore the author prefers to carry an overflow from the pump through a standpipe of sufficient height to discharge into the tank, and by this arrangement the regulation of the

water supply presents no difficulty. The tank, of course, can be drawn upon when the pump is not in use, and if of large capacity is very useful as a standby.

The amount of water required for washing gravel is quite considerable; an ample supply should always be arranged for, as the more copious the water supply the better the results. The pumps for an actual plant very similar to that shown on the diagram were of a capacity of 100 gals. per minute, and the average capacity of the plant was 20 tons per hour, but a great deal more could have been got through had sufficient material been available; the water supply in this case proved to be quite adequate, which would give 300 gals. per hour per ton of material, or, say, 450 gals. per hour per cubic yard of aggregate; the aggregate in this case was river gravel, and in some cases it was fairly clean and in others mixed with a good deal of clay and river silt. In an American plant which came under the author's notice the pump capacity was 450 gals. per hour per ton of material, or, say, 670 gals. per hour per cubic yard of aggregate.

The receiving hopper was about 54 ft. above ground; it was filled in one case by a grab worked by a 100-ft. jib, stiff-leg derrick crane; at this height the crane radius was about 80 ft., and the material was grabbed from a barge whose centre line was approximately 80 ft. from the crane; hence the crane could grab from this barge without luffing the jib; the hopper could equally have been filled from a chain bucket elevator, or by an ash hoist; it pays to get the raw material delivered at a good height in the first case so it can thereafter flow by gravity to its destination.

The hopper was of ample size for two or three grab loads, and the material flowed directly into the screen from the hopper, the water supply being led into the chute; it was deemed advisable to station a lad on this chute to see that no dangerous material found its way into the screen; the author on one occasion had a crusher ruined owing to a railway chair getting in with the gravel supply.

The screen in this particular plant was of the Ord & Maddison internal rejecting type, which is clearly shown by the inner cone and dotted lines in the diagram; this cone is pierced with $\frac{3}{8}$-in. holes, and any stuff over this size slid out through the sloping mouth of the cone into a chute which led directly to the crusher. The material which passed through this cone fell inside the main cylinder of the screen, which was pierced with $\frac{1}{4}$-in. holes, the body of the screen being 10 ft. long; here by the action of the water the gravel was well washed and the smaller sizes from $\frac{1}{4}$ in. downwards

fell through the screen into the sand bin, the sizes $\frac{5}{16}$ in. and over being delivered at the mouth of the screen into the gravel bin. It will be observed that in this arrangement the larger aggregate would heap up under the mouth of the screen, and to fill the bin completely the gravel would have to be trimmed by hand; the labour involved on this was quite small and did not warrant the employment of mechanical means; in cases where it is desired to avoid this hand trimming the author would suggest putting a spiral conveyor along the top of the bin, so that as the gravel heaped up it would be pushed to the far end. Below the sand screen is a water separator consisting of a box like the body of a jubilee wagon, triangular in section and supported on trunnions; the sand and water fell into this box and the water overflowed at the top through a pipe and was led away by means of a gutter which was clear of the short pipe when the box was turned on its trunnions. When the box was full of sand a catch was released and it overturned, emptying into the sand bin below; by trimming the sand with a mud rake as it fell into this box it was possible to fill it perfectly flush with sand before it was upset, but the sand reached the bin in a very saturated condition. This arrangement was not ideal, but very little silt was deposited in the box and the sand was amply clean enough for all practical purposes. As this plant was producing 6 to 8 tons per hour of sand the cost in labour of this sand separator was between 2d. and 3d. per ton, whereas if the sand had been dumped into settling tanks on the ground the cost would have been considerably more than this and valuable space would have been occupied by the settling tanks.

The jaw breaker was of the 20 in. by 9 in. size, and the delivery was taken into a pair of 18 in. by 18 in. rolls set about $\frac{3}{4}$ in. apart to further crush the material which was dumped on the ground, whence it could again be picked up by the grab without altering the radius of the crane.

Where the gravel arrives free from sand, as is sometimes the case with sea and river shingles, it is sometimes necessary to add a smaller pair of crushing rolls to produce the requisite quantity of sand. These are shown at the side of the large aggregate bin, and the delivery from these rolls can be taken through a smaller screen and be graded as required.

The bottom of the bin can be made hopper shaped or flat; if it is made flat the corners cannot, of course, be discharged without trimming, but often flat bins are cheaper.

The lantern slide gives the general idea of the above

gravel-screening plant built in timber for temporary purposes; for permanent purposes reinforced concrete would, of course, be better, and due account must be paid in designing it to the bursting pressure of the gravel.

One lantern slide shows another gravel-screening plant of this type erected by the author during the war, in which the feeding device was on different lines, being in fact in duplicate; a couple of old pile frames were braced together and platform made at the top overhanging the dock, through which a $\frac{3}{4}$ yard skip could be hauled by means of a winch; when the full skip was at the top a flat box running on wheels was run underneath it, into which the bucket was tipped; this box then ran down a slight slope and automatically dumped itself into a chute which led to the screen; this arrangement was rather costly for labour in filling the skips, and subsequently a small grab was purchased which discharged into a hopper, from which the stuff was raised by a bucket elevator into the screen; the crusher was put on to the ground and the rejects were led into it and the discharge into the bucket elevator; underneath the bins was the conveyor belt, which fed two small bins alongside the concrete mixer. One man worked the crane, one man was in the barge, and two men were at the top looking after the sand separator, etc., making a total of four men for an output of about 20 tons per hour, or a wage cost of about 3d. per ton for washing and screening the aggregate. The grab had smooth lips and did not damage the bottoms of the barges at all.

In laying out an aggregate preparing plant it is sometimes found that the natural advantages of the ground can be utilised to save cost of installation and of handling.

The most remarkable case of this that has come to the author's knowledge is the works of The Stone Court Chalk Land & Pier Co., Ltd., situated at Greenhithe, in Kent.

The gravel forms a deposit about 15 ft. thick on top of the chalk; the latter was required for cement-making purposes, and the expense of removing this overburden became a serious question; at first sight the overburden did not appear to have any commercial value, as it was a mixture of earthy material and siliceous water-worn pebble; at some former time, however, a large rectangular shaft had been sunk through this overburden to a depth of some 90 ft. into the solid rock below, and at this depth a tunnel had been driven to connect the bottom of the shaft with the railway siding; the top of the overburden is about 100 ft. above Ordnance datum, and the price of land was too high

to allow of the possibility of dumping this overburden elsewhere; in ordinary quarrying where the quarry-face is developed in the side of a hill it is usually possible to tip the overburden into the bottom of the quarry behind the quarry-face; this was not possible in the present case as the overburden had to be removed before a start could be made, and the bright idea occurred to someone of using the existing shaft as a suitable position for erecting a gravel-preparing plant, into which the material might be dumped from the level of the top of the rock, gradually descending through the various screens and crushers to the storage bins at the bottom, below which the railway wagons can be run to receive the finished product.

The third diagram shows the general lay-out of the plant; the overburden is sufficiently tenacious to be picked up by coarse forks and thrown into 28 cubic ft. jubilee wagons which are drawn by an electric winch to the receiving hopper which is shown on the top left hand corner; at the bottom of this hopper is a mechanical feeding device.

The washer consists of a sheet metal tube about 16 ft. long by 3 ft. diam. inclined at about 15 deg. to the horizontal; the material is fed in at the lower end and the water enters at the upper end, and inside there is a spiral which gradually works the material up the incline against the stream of water; the upper end of this washer consists of three telescopic perforated screens, the inner and longer one of which is perforated for $\frac{3}{4}$ in. material, the next one for $\frac{3}{16}$ in. material, and the outer one of wire mesh for sand.

These three sizes are led by chutes directly into the storage bins at the bottom, while the water from the lower end of this washer is carried away in trough to large settling tanks near the railway siding where the very fine sand is deposited, and can be recovered for plasterers' work.

The rejects from this washer are taken through a crusher and from this crusher it is taken by a breeches chute into two raff wheels; these raff wheels consist of cylindrical drums of wire mesh which allow the $\frac{3}{4}$ in. stuff to pass, but reject anything over this size; they are formed something like a concrete mixer with internal plates which revolve with the drums which carry the material round to the top of the drum, the small stuff falling through and the large stuff falling on to a chute projecting into the drum, and thence into crusher rolls, whence it passes into screens at a lower level, by which it is finally sorted and distributed to the bins.

The water which is used very freely throughout the various processes is drawn from two 7-in. bore holes which

terminate at about 88 ft. below ordnance datum; it is raised from these bore holes by a Sulzer pump with a 6-in. delivery pipe; this pump is driven by a cotton rope from the main shaft placed a little bit below the edge.

The whole plant is driven by a 65 h.p. semi-Diesel oil engine, running at 205 revs. per minute, but the author understands that this engine is powerful enough to drive a similar plant of equal capacity in addition, there being sufficient room in the shaft to erect a twin plant.

It will be noted that no attempt is made to separate the sand and the water, and that both flow together into the sand bin.

As an interesting example of a plant for dealing with river gravels the author is permitted to give a description of the well known works of the Ham River Grit Co.

These works are situated at Ham, near Richmond, on the south side of the Thames; they consist of a large tract of land, having a gravel bed 15 to 20 ft. thick, the bottom of which is below low water level of the Thames, under which is the clay; an entrance has been cut to the river through which barges can be taken; the river is tidal, and at low water the depth above the clay is not sufficient to float the large crane pontoons, and to remedy this a new entrance is being constructed to the river, with lock gates, by which the water can be impounded and the craft kept afloat at all hours of the day.

There is a very thin overburden of sand and vegetable matter which is stripped by hand.

The gravel is won by means of grabbing cranes carried on broad-beam pontoons, the grabs being 1 cub. yard capacity and of the single chain type; the raw gravel is grabbed from the toe of the gravel face, where it lies in a loose heap, having fallen by its own weight and the action of the water; smooth lipped grabs are employed, and with a smart man on the crane a cubic yard can be picked up and discharged in from 30 to 40 seconds, so that each crane is capable of an output of somewhere about 100 cubic yards per hour; holes are cut in the grab to allow the water to escape, and when properly placed, the majority of the water is discharged, while the crane is swinging through the 180 degrees from the bank to the barge; some water is, of course contained in the gravel which is pumped out by one man on the bilge pump of the barge, but it is obvious that the costs of getting could not be much reduced; the barges which hold about 190 tons are brought alongside another grab-crane which unloads them direct into the gravel screening and washing plant.

This plant is of the simplest design imaginable, and as far as the author can see performs its purpose with the greatest efficiency.

A chute leads from the discharging hopper to the top of an inclined flat plate jigging screen; the gravel is assisted down the chute by a jet of water and slides on to the top section of the screen which is perforated with $\frac{1}{4}$-in. holes. This screen is merely a timber frame with 10 in. by 4 in. side timbers on edge placed about 6 ft. apart and about 16 ft. long. These timbers are supported on trunnions at the upper end and by spiral springs about two-thirds of the way down; below these springs is a revolving shaft set across the screen on which are four striker arms which strike appropiate plates placed on top of the frame timbers; at each revolution of the shaft the screen receives four blows which stretch the springs and give an up and down motion to the screen; as the screen is set at an angle of about 25 to 30 degrees to the horizon, this jigging motion causes the gravel to slide down at a slow rate, and the material is screened by falling through the appropriate holes. The screen plates form the bottom of the timber frame, and are about 6 ft. wide by about 4 ft. long; the top plates are pierced with $\frac{1}{4}$-in. holes, the next row with $\frac{5}{8}$-in., and the bottom with 1 in. holes; where the screen plates change they overlap each other with a slight fall of perhaps 1 in., allowing the material to drop and turn over as it falls. Water jets play on the gravel where necessary. Repairs to this screen are easily effected either by renewing the plates or by bolting a patch over any worn holes; such a screen will give an output of 30 to 40 tons per hour of perfectly assorted material.

The oversizes are delivered at the bottom of the screen down a chute to a Hecla crusher with 24 in. discs, and from this crusher the crushed material is taken up by a bucket elevator to the screens once more.

The bucket elevators are of very simple design, consisting of flat bars 1 in. wide, each end of which is bent round a mandrel to form an eye; four or five of these flat links are assembled with a pin at each end, and spaces between each link of the same width as the link, to form one section of the chain, and the next section, of course, have the links placed alternately with those of the preceding ones; the links lie on the flat, not on their edges, and the buckets are bolted directly to them. The links are of mild steel, and the wear is stated not to be excessive.

The whole plant has been designed for continuous work, and it has been found better to have simple cheap plant which can be scrapped when unduly worn rather than expensive plant which keeps everything at a standstill when it is undergoing repairs.

The material dropping from the screen forms heaps which are picked up by grabbing cranes and loaded into barges. This, again, is a very inexpensive process when large quantities are being handled and allows for a thorough separation of the water while the heaps rest on the ground.

The only simplification that appears at all possible in the above processes has already been undertaken by the company, and it follows on lines suggested by the author to another gravel company some years ago.

This simplification consists in putting the grabbing unit and the screening and crushing units on the same floating pontoon, and in delivering the prepared material over the side into barges which take it directly away to the market.

The lantern slide was taken from a photograph of this floating pontoon; at one end is an electric grabbing crane, the grab from which delivers into an elevated hopper; next to this is placed the motor house containing the motor, switchboard, etc., for driving the whole plant; from the overhead hoppers the raw gravel is screened by two sets of jigging screens, similar to those described above, and a Hadfield's Hecla crusher deals with the rejects which are again elevated and screened.

The screen product is elevated by bucket elevators into small storage hoppers from whence chutes are taken over the side for filling the material into barges.

The Ham River Grit Co. at one time employed a floating gravel pump for winning the raw material.

Another lantern slide shows this floating pump; the plant consists of a belt driven centrifugal pump, engine and boiler mounted on a small pontoon; a suction pipe is taken to the bottom of the toe of the gravel face, and the rapid stream of water draws the sand and gravel through the pump to the discharge; this can either be on land or into a barge with watertight bulkheads; the water and gravel fill the barge and the water gradually overflows the coamings, leaving a barge load of very wet gravel. This method of getting gravel is obviously very expensive on account of the horsepower required, and of the trouble with the pipe lines, etc.; the wear on the pump is very great, and the author is informed that only about 15 per cent. of the discharge is gravel.

It seems to be a very different proposition to pump gravel in this way than to dredge sand and silt from harbours, and the author is unable to understand why gravel pumps of this type are fairly common in America.

As an example of the handling and preparation of pit gravels the author is permitted to describe the works of The Yorkshire Amalgamated Products, Ltd., of Doncaster, who have two extensive gravel pits for the supply of aggregate for concrete, one being situated at Bawtry and the other at Barnby Dun, both near Doncaster; the gravel at both these pits is very much of the same nature, consisting of inclined strata of sand or of sandy gravels of very varying thickness; sometimes there are beds of pure sand many feet in thickness, and sometimes beds of pebble mixed with sand, the pebble being in some cases several inches in diameter; the pebble is very largely quartzite, which is an extremely hard and abrasive material and very difficult to crush; it is so hard, in fact, that it scratches glass easily. The last lantern slide shows the gravel face in the Bawtry quarries; here the gravel is won by means of steam grabs; there is a thin overburden consisting of vegetable soil, which is stripped some distance back from the working face by means of a grab worked by a crane with a very long jib; when the overburden is removed the gravel is picked up from the foot of the working face by the grabs and dumped into side-tipping wagons, holding perhaps 3 cubic yards each; these wagons are carried on rather lofty carriages so that when side tipped they discharge at a good height; the gravel is fairly hard packed and stands with a vertical face, so that it has to be undercut by picks in order to produce a fall which the grab can deal with; this serves to fairly mix the different strata so that a uniform product is got; a locomotive takes the wagons to the foot of a bucket elevator, which discharges the gravel at the head of an inclined chute about 2 ft. 6 in. wide, the bottom of which is formed of piano-wire screens, the whole chute being set at an angle of about 40 degrees; each screen is about 5 ft. long, the top one being set with wires close enough together to let the fine sand pass, the next one the pea gravel, and so on; light swinging baffle plates of iron are placed close to the piano wires to prevent the material reaching too great a velocity; the whole arrangement seemed to be fairly efficient, and the gravel was dry enough to pass through the screens easily; the pea gravel, however, was delivered rather mixed with sand, and for special purposes was rescreened in order to rid it of this

sand; the rejects from the bottom of the screen were elevated by a textile belt bucket elevator to a Hadfields Disc crusher placed at the top of the building; these rejects consisted largely of quartzite which was found to be too hard for ordinary crushing rolls, but was very effectively dealt with by this Disc crusher; from this disc crusher a further set of wire screens separated the material into various grades, the finest of which was used at the Bawtry works for making sand bricks for lining coke ovens, etc., while the coarse aggregate was sold for reinforced concrete, and the Writer was informed that concrete made from this aggregate was found to be very impermeable, the large cooling tank at Messrs. Vickers' works having been made from it with most satisfactory results; this tank is 120 ft. long by 50 ft. wide by 50 ft. deep.

In the Barnby Dun Works of this company the gravel is dug entirely by hand; the gravel face is 10 to 15 ft. high, and light Jubilee rails are laid close to the face; a gang of four men can fill 25 three-quarter cubic wagons per hour, or, say, $6\frac{1}{4}$ tons per man-hour; the trucks are pulled by horses to the foot of the bucket elevator, which raises the material to the screens, the material being simply dumped into a chute, whence it flows into the buckets; the screens are of the chain-hung Ord & Maddison type, and special arrangements have been made to keep the sand and water off the chains, thereby greatly prolonging their life; a plentiful supply of water is used and the sand and water run off to large settling tanks of timber; in these the water deposits the sand and flows off with the mud and silt to the settling pond, from whence the water can be used over and over again when it has deposited the silt.

The arrangement of the screening unit for a gravel plant requires considerable care and thought if labour costs have to be kept low. The elementary screen is the ordinary builders' screen, consisting of a wooden frame about 5 ft. long by 2 ft. 6 in. wide, covered with parallel wires set a certain distance apart and kept in position by cross-wires placed 3 or 4 in. apart; this is set up on end at a fairly steep slope, and the material is thrown into it by shovels, the fine stuff passing through and the coarse stuff falling to the bottom. The results obtained are very inferior and very costly; if the amount of sand in the raw material is small a great deal of labour is incurred in repeatedly throwing the coarse stuff at the screen; if on the contrary the percentage of large pieces is small the cost per cubic yard of screening is reduced because the majority of the stuff will pass the

screen at once. The next development of this screen is the ordinary grizzly, which forms a part of many mechanical screening appliances. This consists of an assemblage of iron bars, commonly rectangular in section, with a depth much greater than the width; the spaces between these bars is arranged so that the material of the desired size shall fall through, the larger material rolling to the bottom and usually being led off into a crusher; these bars are inclined at about 45 degrees to the horizontal, but a much better plan is to hinge them at the bottom so that the angle can be varied to suit the material; if the material is fed on to the grizzly with a plentiful supply of water quite good results can be obtained.

In the ordinary type of fixed bar screen there is usually a sheet-iron chute below the screen into which the water and material passing through the screen are received, and which delivers them on to a second screen, which removes another grade, and so on; by arranging the bars so as to pick out the coarsest material first and by introducing a plentiful supply of water over each screen you get what is most desirable, namely, that the fine material gets the most washing.

The defects of the fixed bar gravity screen are due to the fact that the material usually acquires too great a velocity in sliding down the screen, so that the small sizes do not pass through; any attempt to check the velocity by cross-bars is useless, as it causes the pebbles to jump; if the inclination is so gentle that the material passes down at a very slow rate it will pack and stick, although a plentiful supply of water minimises this difficulty. Hence these screens, while being very useful for picking out over-sizes from a quantity of granular material, are not very useful in grading the material itself, unless it consists of mixtures of material of uniform size, such as, say, of sand $\frac{1}{16}$ grains, pea gravel $\frac{5}{16}$ grains, and stone of, say, $\frac{3}{4}$ grain; piano-wire screens set $\frac{1}{4}$ in. and $\frac{3}{8}$ apart would grade this material fairly efficiently.

To remedy this defect these fixed bar screens are sometimes placed at a fairly flat slope and jigged or vibrated; the slope must be sufficiently flat to prevent the material sliding down when the machines are at rest and steep enough that the material will have a uniform velocity of about 1 ft. per second when the screens are jigged or dithered; a screen of this type has already been described in a previous part of this paper.

The most commonly-used type of screen is undoubtedly the revolving cylindrical screen. This consists of a metal

cylinder perforated with the appropriate-sized holes and revolving at a fairly slow rate about its axis, which is inclined at a slight angle to the horizontal. The screen may be made of mild steel plate, or cast manganese steel plates, or woven wire mesh; the screen may be perforated with holes of one size from beginning to end, or it may be divided so as to sort the material into various sizes, the first portion being pierced with fine holes, the next with coarser, and so on; the holes should in all cases be slightly larger than the intended size of the material to allow for irregularities. It will be noticed that the fine material is eliminated first, and so on, in ascending degrees of coarseness; this is a drawback where the material has to be washed, because the fine material needs most washing, but in this arrangement it is carried through at once with the water, and so does not get thoroughly washed.

A better arrangement, therefore, is the American plan of using conical screens assembled one after the other at some little distance apart, on a long shaft, which is set at an angle of about 30 degrees to the horizontal; the wider end of each screen is uppermost, and the first screen has the largest holes, and passes, therefore, all the material except the oversize, which falls out at the lower end of the screen; the stuff which passes through the screen, mixed with washing water, is received in a steel trough, down which it slides and enters the larger end of the screen next below; this is perforated with holes of the next size smaller, and the material is usually subject to a further washing; a third screen may follow the second one, and so on, according to the number of sizes of material it is intended to produce This type of screen gives the most efficient washing possible, and is undoubtedly superior in this respect to the ordinary cylindrical screen.

Reverting to the latter, however, it may be remarked that it is by no means necessary to have one long screen, as the cylinders with the various size perforations can be placed one inside the other by making them of different diameters; in this case the smallest screen has the biggest holes, and it it also made slightly longer than the next size, and so on, in order to allow of the collection of the rejects from each screen in suitable shutes. Screens of this kind are not usually used in gravel preparing, and they have this drawback, that the discharge of each kind of material occurs at about the same spot, thus making the distribution into the various bins a matter of difficulty; in fact, a long cylindrical screen not only screens the material, but also acts as a distributor to

carry it over considerable distances and to deposit in the respective bins, so that it is sometimes desirable to chose a screen of comparatively small diameter and considerable length in order to fill the respective bins.

It will be noticed that in a cylindrical screen as above described the fine material is deposited near the inlet end, and the coarser material farther away until the rejects are delivered at the extreme far end; as these rejects commonly have to be crushed and elevated again to the screen this delivery at the far end becomes a drawback; to overcome this Messrs. Ord & Maddison, Ltd., of Darlington, have brought out their internal rejecting screen, which consists of a screen of fairly large diameter, at the receiving end of which is an internal cone, tapering inwards from the mouth of the screen and forming in itself a conical screen with perforations sufficiently big to allow all the materials to pass except the oversizes which slide down to the mouth of the screen, and are taken to the crusher; the body of the screen can be arranged in the ordinary way commencing with small holes and going on with larger ones; screens arranged in this way must be of large diameter in order to get sufficient surface in the internal cone to properly screen out the large pieces without rejecting any of the desired size; the material, too, must be delivered at the far end of the internal cone, and if it is at all sticky it is sometimes difficult to get the chutes at a flat enough slope to do this, and at the same time to allow the material to move by gravity down these chutes; with fairly clean gravel, however, this screen is extremely useful. Messrs. Ord & Maddison also depart from ordinary practice in that they hang their screens in sling chains which pass over sprocket wheels above and round grooves formed in the screens below; when the sprocket wheels are turned the screen, of course, revolves, and as it is set at an incline there is a thrust screw at the bottom end pressing against the spider plate which keeps the screen in its proper position. This arrangement of chains allows the mouth of the screen to be quite open for the introduction of the material, and this is an important point.

Sometimes screens are carried on central shafts and sometimes on external rollers; the latter arrangement seems preferable, and in all cases the receiving end cannot have spokes, or the chute would be unable to enter. When the screen is supported on exterior rollers it is usually revolved by spur or bevel gearing, which should be very carefully arranged so as to keep it as far away as possible from the

fine grit which otherwise causes rapid wear. The same attention must be bestowed upon the support at the receiving end which commonly wear rapidly from this cause; even the chain suspension of Messrs. Ord & Maddison is not free from this defect as the author has found that the chains wear more rapidly at the receiving and allowing the screen to drop and lose its proper slope. This, however, is easily remedied by removing a couple of links, the chains, which are of $\frac{3}{4}$-in. oval linked patern, are supplied with a split link which can be easily undone when two or four links can be sawn out and the chain rejoined; as a matter of fact, an ordinary link with a $\frac{3}{4}$-in. gap cut through one side makes a quite efficient joining link.

The next part of importance in an aggregate preparing plant is the crushing unit. There are five types of machines ordinarily used for coarse crushing. These types are the jaw crusher, or breaker, the gyratory crusher, the disc crusher, the roll crusher, and finally the edge runner; material reduced by the above crushers can be further reduced in ball and tube mills, as for cement and paint, or by impact crushers, as for flour, chemicals, etc., but the concrete engineer never has to concern himself with the latter types.

The jaw breaker, or jaw crusher, is known to everyone, but to those who have not had to deal with it a few remarks may not be out of place. It consists essentially of a heavy cast iron or cast steel box-shaped frame, to one end of which is secured a removable hard casting forming one of the jaws; a second jaw consisting of a very heavy casting, with a removable face, is hung parallel to the first jaw from a shaft about which it can swing; the space between these two jaws at the upper end gives the size of the crusher; thus a crusher 10 x 6 has jaws 10 in. wide by 6 in. apart at the top: the bottom end of the swinging jaw approaches very nearly to the fixed jaw, and the distance apart here gives practically the size of the product; behind the swinging jaw is the main shaft of the machine; between the two bearings this shaft is formed into one continuous eccentric, which gives a small up and down movement to what is known as the pitman; this pitman is a heavy casting hanging vertically from the eccentric. At the lower end of the pitman is an abutment or housing for two toggle plates, one on each side of the pitman; the toggle plate at the back takes a bearing against the back of the crusher while the toggle plate at the front bears against the lower end of the swinging jaw. It will

really be seen that as the pitman rises and falls these toggle plates swing the jaw backwards and forwards with an almost irresistible force; a tie rod is attached to the bottom of the swinging jaw, and by means of an adjustable spring at the back of the crusher keeps the jaw toggles and pitman firmly bound together; wedges behind the back toggle plate adjust the opening at the bottom of the crusher and take up wear.

Diagram No. 4 shows a jaw breaker supplied by Messrs. Joseph Booth & Sons, of Rodley, near Leeds, to whom the author is indebted for the wall diagram. In this crusher the pitman is kept unusually short and light, and the whole arrangement is very compact and neat.

The movable jaw plates of the crusher are made either of hard chilled cast iron, or manganese steel, and they can be either fluted or plain; the wear and tear on these jaws is very high when dealing with river gravels, and the manganese steel plates undoubtedly last much longer than the cast iron; there is, however, not much advantage in their use owing to their greater cost, although the delays incurred by the more frequent renewals of the cast iron jaws add appreciably to the cost of the latter. To get the maximum work out of a crusher the feed should be very regular at about the maximum capacity which the crusher can deal with; the delivery from the bottom of the crusher should be arranged so that by no possibility can the crusher choke up, or serious damage will be done; if the material is led away by a chute the crusher should deliver on to this chute in such a way that should this chute choke up at the lower end the crusher can overflow at the top end without choking up; this is a most important precaution and very often neglected. A jaw crusher should never be expected to give a large quantity of fine material; it will always produce a certain quantity of fine material, and the smaller the crusher the bigger is this quantity; but to produce fine material, say, $\frac{3}{4}$ in. stuff, in large quantity recourse must be had to crushing rolls; in fact the jaw crusher is really a breaker to break up large masses of rock or stone.

In mounting jaw breakers on elevated bins, etc., ample means should be taken to provide against their vibration; the speed of crushers varies from 250 for small crushers to 160 for large crushers, and owing to the arrangement of the toggles the jaws usually make two swings per revolution; these jaws are fairly heavy and their momentum has to be taken down to the ground by suitable means; it is therefore advisable to mount them on the ground wherever possible.

The second form of crusher is the gyratory crusher, which is not so well known as the jaw crusher; it consists of a vertical cylindrical casting, at the top end of which is secured a truncated cone with its wide end uppermost, inside this cone hangs a second cone with its wide end downwards; this second cone forms the continuation of a hollow shaft which goes to the bottom of the machine, the shaft being continued above the cone, and held in a spherical bearing carried by a bridge across the mouth of the machine; at the bottom of the machine is a mitre wheel whose elongated projecting boss runs inside a suitable bearing; this boss receives the lower end of the shaft which carries the inner cone in a hole which is bored rather out of the centre of the boss; thus when the mitre wheel revolves the shaft is carried round like a conical pendulum, the apex being at the top of the crusher; by this means the small inverted cone swings round inside the larger cone in a conical path, and thus gives a crushing action to any material introduced between the two cones; neither of the cones revolve, however, and the crushing action is very clean without undue abrasion. The cones are usually made of manganese steel and are comparatively thin.

The disc or plate crusher is very similar to the gyratory crusher in principle, but very different in construction; in these the crushing is done between two saucer shaped discs of manganese steel; these run on a horizontal axis, and the outer one is perforated for the introduction of the material; they are placed with their cavities together and their edges nearly touching, thus forming a crushing chamber; the axis on which they revolve are not in line nor parallel, but are at a slight angle, one saucer in fact being carried on a shaft, the far end of which is carried round in a circular path, so that this shaft, too, forms a conical pendulum, the axis of the cone being horizontal; this saucer works in a hollow cup which forms a backing to resist the crushing strains, and it will be readily seen that the circular path of the outer end of its shaft will cause the lip of this saucer to approach the other saucer with a kind of rolling motion; the second saucer is revolved by a belt pulley at speeds varying from 100 to 200 revolutions per minute, according to the size of the crusher, while the pendulum shaft is driven at twice this speed; when material is introduced between the two saucers both speedily assume the same rate of revolution, and the material is crushed by the rolling motion of the inner saucer, being flung outward by centrifugal force. These

crushers are particularly suitable for dealing with hard pebble which they will readily reduce to pass a $\frac{3}{4}$ ring, but some consider that their output is too low; the author very much doubts, however, if they can be beaten by any other crusher for their particular class of work; great ingenuity and skill has been shown in the design of the machine by Messrs. Hadfields, Ltd., of Sheffield, who are the sole makers in England, and to whose courtesy the author is indebted for the lantern slides. The base of the machine is formed into a large oil well and a belt driven pump keeps all bearings properly lubricated, which is an approach to motor car practice not usually found in contractors' plant.

The above forms of crushers or breakers will not produce the bulk of the large aggregate required for concrete, except, perhaps, the disc crusher, last described. After leaving the crushers, therefore, it is usually necessary to take the stone through crushing rolls; these are very simple in principle, but call for much care and skill in the design; essentially they consist of two metal cylinders revolving on parallel axes placed in the same horizontal plane: the circumference of the cylinders do not touch each other, but a gap is left between them of width according to the size of the material desired; the cylinders revolve towards each other, i.e., the left one clockwise and the right one counter clockwise, so that any material thrown on to the top of the cylinders tends to be drawn between the cylinders and to be crushed in passing. These cylinders or rolls revolve in bearings moving in horizontal guides and controlled by screws, wedges or springs; the great risk in roll crushers is that of some uncrushable material, such as iron, causing damage by getting between them; hence many makers keep the rolls up to their work by exceedingly strong spiral springs; the drawback to these, however, is that they allow the rolls to give when crushing ordinary stone, so that a uniform size of product is not obtained; another scheme to avoid damage is to hold them up to their work by screws which pass through cast-iron lugs secured to the frame: these lugs break off if any undue stresses occur and can be easily renewed; a better plan is to employ a magnetic separator to remove any possible vagrant iron, which is the only material likely to cause damage to the rolls; sometimes the rolls drive each other through long-leafed pinions, in which case they are usually slow-running rolls, and sometimes each roll is driven by its own heavy-belted flywheel, one of the belts being crossed in order to give the contrary direction; in this case

the speed is rather high, and with these belt-driven rolls there is less danger of damage through vagrant iron; the shell of the rolls is usually in manganese steel wedged on to a cast-iron core and run in with lead; these shells wear rather rapidly with gravels and other abrasive materials, and it is, therefore, advisable, in order to avoid delays, to keep a spare pair of shafts with shells ready mounted to slip in when required. The feeds through the rolls should be carefully regulated; the maximum capacity of the rolls is, of course, the area of the opening between them multiplied by the circumferential speed, and it would be very interesting to know what is the average amount of voids in this volume for crushing rolls as ordinarily arranged.

For instance, a pair of 18-in. roll crushers with their rims set $\frac{3}{4}$ in. apart would have an opening between them of about 1 12th square ft. area; at 60 revs. per minute this would give a crushing volume of 60×18 in. $\times 3.14 \times 18$ in. $\times$ $\frac{3}{4}$ in. $= 24$ cubic ft. nearly of crushing space per minute; if this could be completely filled with stone these crushers would give an output of about 100 tons of crushed stone per hour, whereas the working output of such would probably not exceed 6 tons per hour. In order to obtain the maximum output through crusher rolls for flint gravel the author constructed a small hopper over the rolls, the sides of which were cut to fit the curves of the rolls exactly, the curved surfaces of the rolls themselves forming the bottom; this hopper was kept full of clean $\frac{3}{4}$ in. pebble, and under this arrangement about 8 tons per hour were passed through a pair of 12×6 rolls running at 60 revs. per minute; the product was crushed from $\frac{5}{16}$ down to very fine sand. This will give a maximum crushing capacity of about 16 tons per hour, as against the 8 tons actually obtained. The author believes that roll crushers arranged in this fashion might prove very useful in the preparation of sand from gravels, but the rolls have to be of very robust design.

Diagram 5 shows a general arrangement of a pair of spring control crusher rolls made by Messrs. Edgar Allen & Sons, Ltd., of Sheffield, to whom the author is indebted for the diagram; these rolls are gear driven and kept up to their work by two pairs of coiled springs; they have removable manganese steel shells, which can be renewed when wear takes place.

In all the foregoing types of crushers it is desirable not to attempt to reduce the material too greatly at a single crushing; if the material is very large and it has to be greatly reduced the operation should be done in two, three or more stages, reducing at each stage to, say, 1/6th or 1/5th.

For producing sand from sandstone, or in crushing clinker, the familiar edge runner or mortar mill is the most suitable form of crusher; it consists of a heavy shallow metal pan revolving on a vertical axis with two heavy cast iron rollers revolving on a circular track on its bottom; the bottom is pierced with holes of the size necessary for the intended product, and as the pan revolves the stone or clinker is forced by centrifugal force or by guide blades into the track of the rolls and crushed by their weight.

The author has seen these used for crushing blast furnace slag in a concrete products factory with success.

The lantern slide illustrates one of these edge runners manufactured by Messrs. Hadfields, Ltd., of Sheffield.

There are two items of plant connected with crushing and screening which deserve some attention, namely, mechanical feeding appliances for feeding crushers, screens, etc., and water separators.

A simple form of mechanical feeder takes the form of a circular table revolving on a vertical axis; the material is delivered on to this table from a spout placed directly over, and at some distance above, the centre where it forms a cone, the sides of which have a certain definite slope, and this cone revolves with the table; a vertical cutter is adjusted so as to cut into this cone, and the material outside this cutter is thereby forced off the table into the hopper below; by adjusting the cutter a greater or less speed is obtained; these mechanical feeds can be used for gravel or soft material like gypsum. For hard material, such as lump limestone, the tray feed is better; this consists of a flat reciprocating tray with sides placed below the hopper mouth; this hopper mouth is rectangular on plan, two of the said sides and the back being brought down to the level of the tray, while the fourth side or mouth terminates at a sufficient distance above the tray to let the largest lumps pass through; as the tray is pushed forwards it carries the material with it and fresh material descends; the tray then moves back and slides under the material; at a fresh forward stroke more material descends, and so on, until the material finally falls over the front edge of the tray; this, of course, gives an intermittent delivery.

A form of mechanical feed very common in America is an articulated metal belt carried on two pulleys, something like a caterpillar tractor; this is placed directly under the hopper mouth and gives a continuous feed.

For small stone of uniform size and sand the author frequently employs a long hinged chute on to which the bin opening delivers; by making the slope of this chute steeper than the natural angle of the material the latter flows down at a certain speed varying with the slope; by raising the chute the delivery stops instantly; the apparatus is very easy to construct, and gives very good results with washed gravel of uniform size; with sand, however, the natural angle of which varies so much with its moisture, the results are not so satisfactory.

As regards water separators, the author has already described a simple easily made separator; in America a very common form of these is a truncated cone of sheet metal, the lower end of which is closed by a disc which is held to a fixed beam, the cone itself being counterpoised; sand and water flow in at the top and the water overflows, carrying with it the mud; when it is full of sand the cone drops for some distance and allows the sand to flow out past the valve.

Another and very ingenious form of water separator is that patented by Mr. Rikoff, and marketed by the Perfect Concrete Mixer Co., of Broxbourne. It consists of a wide angled cone made of sheet metal revolving on a horizontal axis; a number of troughs are fixed to the outside of the cone, running from the base to the apex; it will be seen that as the cone revolves that particular trough which is on the horizontal diameter of the cone is perfectly level from end to end, but that in a quarter of a revolution this trough will have a fall towards the apex of the cone depending on the taper of the cone; if water and sand are introduced into this trough when it is horizontal the water will be run off as the trough approaches to the vertical diameter of the cone leaving the sand behind; in another quarter of a revolution the same trough will be in a horizontal plane once more, but will be inverted allowing the sand to drop out.

The lantern slide shows this separator working in conjunction with a sand and gravel washer and screener supplied by the Perfect Concrete Mixer Co. These separators very effectively separate the sand and water, provided the speed is carefully adjusted for the class of sand being used.

As regards plant for digging gravels, two very useful forms of these are first the grab and second the dragline excavator.

The former is very well known in England, but a brief description may not be out of place. Essentially the grab

consists of a semi-cylindrical shell with end plates, divided down the medium line into two cylindrical quadrants; these are hinged together by means of the end plates at the axis of the cylinder, and their outer lips are held by two steel arms to a cross beam running above and parallel to the axis; it is obvious that the two quadrants will open and close on the medium line by making this cross beam, and the axis approach or recede from each other; the crane rope which lifts the grab passes round a system of pulleys fixed alternatively to the beam and to the grab axis, thus drawing the beam and the axis together with a force equal to the weight of the grab multiplied by the number of times the rope passes round the pulleys.

Hence the force with which a grab will close is very considerable; when it is desired to open the grab the beam is held suspended while the lifting rope is paid out, so that the weight of the grab itself forces the two quadrants apart.

In the single rope grab there is a hook in the top beam which catches on a ring suspended from the top of the jib, and when the grab is hanging from this hook it will open out and discharge if the lifting rope is paid out; in the two rope grab the beam is itself carried by a second rope passing over a second jib head pulley and worked by an independent drum on the crane; in a properly designed grabbing crane the second barrel is driven from the main engine or motor and the control gears are so designed that the operator can work the crane with great rapidity; these grabbing cranes are very substantially designed so as to stand the shocks and strains incidental to grabbing; they are also high speed because a large grab may require 20 or 30 ft. of rope to be wound in, in order to close the grab before any lifting takes place; thus the grab itself may have to be lifted only 10 ft., but perhaps 20 ft. of rope has to be wound in before the grab is closed so that the crane has to wind in 30 ft. of rope each time; hence a slow running crane is very undesirable. If it be desired to work a two roped grab from an ordinary crane the makers can supply a powerful coil-spring drum for working the second rope; this drum merely keeps the opening rope taut; when it is desired to open the grab this drum is braked and the main drum is allowed to pay out.

In choosing the size of grab for any particular output regard must be had to the time occupied in adjusting the

grab over the material and over the hopper; the former operation when grabbing from a barge or railway truck is always much more lengthy than one would imagine. Another source of annoyance in using grabs with gravel is the presence of exceptionally large flint stones in the gravel, which sometimes get between the jaws and allow the contents to flow out as the grab is lifting; when grabbing from a natural gravel bed the nuisance from this is not very serious, but from a barge or railway wagon it may mean losing the gravel on the quay and causing a nuisance; with single chain grabs the author has found the best way of dealing with these is to knock them out of the jaws with a 7-lb. hammer, although this is apt to cause severe shocks to the crane; the author has never known any harm happen, however, if the grabs are worked by a rope on the crane instead of a chain; with two chain grabs the grab can easily be opened to get rid of the obstacle, but even here with gravels containing a fair proportion of large-size flints this is a nuisance which has to be reckoned with. Grabs for digging hard packed material should be of the whole-time variety; that is, the cylindrical shell is placed by a series of massive steel bars which dig into the material, whereas for loose material plate grabs are amply sufficient.

Grabs are designed for a large variety of purposes, from lifting heavy iron ore down to light grabs for grain handling, and the weight of the grab itself is usually rather more than the weight of the maximum load it will lift; consequently it must be specially designed for its purpose, and considerable experience is necessary; the author is indebted to Messrs. Priestman for the lantern slides and for the loan of the model, which may be interesting to those who have not yet had grabs under their personal control.

The dragline is very little known in England at present, and the author was the first to introduce it here, as it is particularly suitable for digging gravel for concrete purposes. It consists of a cableway, one end of which is supported on a lofty mast and the other end of which is anchored to the ground 500 or 600 ft. away; a massive steel scoop runs on this cableway; the track cable terminates at the mast in one of a pair of three sheave blocks, round which a tension cable passes; by slackening out this tension cable the track cable can be made to touch the ground at any point where the bucket happens to be; a tractor cable is secured to the bucket and passes over the pulley at the top of the mast on to a two-drum winding engine; the second drum takes the

before-mentioned tension cable; imagine now the bucket is resting on the ground which it is desired to excavate, and that the tractor cable be pulled in; the bucket forces itself into the material, being provided at its front edge with manganese steel teeth, and speedily fills itself; when it is nearly full the second drum of the winding engine is put into motion, thus winding in the tension cables and lifting the bucket out of the ground; as the bucket leaves the ground its speed is increased, and it is hauled rapidly up the slope of the track cable to the top of the mast; the bucket is hung from a carriage by an arrangement of pulleys and chains in such a way that when it meets with a stop clamped to the track cable it is up-ended and made to discharge its contents into the gravel-preparing plant; it is then allowed to run down the slope of the cable gravity, and it is stopped at the digging spot. The buckets are made of various capacities from $\frac{1}{4}$ yard to 2 yards, and the span may be anything up to 900 ft.; the height of the mast should be 1/7th of the span, and masts are usually in steel for the heavier equipment, although for smaller spans and equipment a mast of trussed timber will suffice; the masts usually require six guy ropes, preferably at an angle of 45 degrees to the ground, and the anchorages need very careful attention Where the deposits are shallow it is preferable to anchor the outward end of the track cable to a bridle cable, the anchorages of which are perhaps 120 ft. apart; the track cable can be made to travel along this bridle cable by a pulley block arrangement, so that this track cable can operate over a segment of a circle having a chord of 120 ft., so that an enormous quantity of stuff can be dug without fresh anchorages.

For the actual digging one man at the winch is sufficient, and 30 to 40 trips per hour can be made, forming, therefore, one of the most economical ways possible of working a sand or gravel deposit, the gravel being delivered automatically into the screening and washing plant.

The author has brought a small working model showing the principle of this plant, and it is also illustrated by the lantern slide.

The above descriptions of the plants are necessarily brief, and it will always be found that each particular proposition must be dealt with on its own merits; this is especially the case when preparing gravels for reinforced concrete, as these vary so tremendously in their composition; in some cases they are accompanied with such a large amount of earthy matter that the problem of washing becomes a very

serious one, and special washers have to be designed, so that care and experience are necessary to ensure economical results.

DISCUSSION.

Professor F. C. Lea opened the discussion, taking the occasion also to thank the Institute for the presentation of its medal. He said that he felt it a great honour to attend and receive the medal which had very kindly been given to him. At the time when he gave the paper, the meeting had very gracefully passed a hearty vote of thanks, but perhaps he might be pardoned for saying that he felt that the medal in his pocket was a little more concrete expression of thanks to him for the work he had done in preparing the paper. A great deal had been said about that paper at the time of his reading it, but he could not let the present occasion go by without saying another word. The Chairman had very kindly mentioned the little physical disability which he (Professor Lea) had suffered at the time. He had taken many thousands of observations for that paper, but had been at the same time preparing a paper for another Institution. He dared to say that some of those present had seen that the Institution of Civil Engineers had honoured him this year in the same way as the Concrete Institute had done.

Referring then to Mr. Sargeant's paper, Professor Lea said he was very pleased to be present and hear it read. He said that most of those who dealt with aggregates in the making of concrete were apt to think that the crushing and preparing of those aggregates was a matter that did not concern them very much. But he was sure that it did. In the first place, methods of manufacture must mean a very great difference in the cost, and that meant a great deal in the preparation of the designs, and the possibility of those designs being carried out. The next point was one affecting many of the results that he had given in his own paper. He had given a great deal of data with respect to the strength of gravel aggregate concretes, and certain of their properties. But the point he wanted to refer to now, was a very important one that had been mentioned by the author of the paper. It was that when one was preparing gravels, especially if they were pit gravels, it was very necessary that a very large quantity of water should be used in order that they might be perfectly washed. A few weeks previously it had happened that he was called in to advise as to a very

large building where very large span girders, or rather beams, were being used. It was a difficult job, and the material which was being delivered on the ground was gravel. Certain blocks had been prepared with great care, but instead of giving such results as he had mentioned in his paper, the results had been about six hundred to seven hundred pounds per square inch for the crushing of the concrete. Fortunately this had been discovered when only very little work had been done. The cause was simply that the gravel was dirty, and he would like, as one interested in the subject of concrete, to say how much he had appreciated the remark in Mr. Sargeant's paper as to the necessity of a large quantity of water. It was a very wise thing for Mr. Sargeant to say that plenty of water must be used in the preparation of gravel aggregate, for the cleaning of those aggregates was of the utmost importance.

Professor Lea concluded by again thanking the Institute for the honour it had done him in presenting him with the medal. He also thanked Mr. Sargeant for his paper.

Mr. A. H. Barnes moved a very hearty vote of thanks to the lecturer. He said the paper had been a very enlightening one, and they could all see that it came from one who had a very broad view of the subject he was handling—much broader than that of a great many other people. He (Mr. Barnes) had enjoyed the reading of the paper very much, and had pleasure in proposing a hearty vote of thanks.

Mr. H. K. Bamber seconded the motion for a vote of thanks. He said he appreciated the importance of the subject and he spoke as one connected in a somewhat large way with the cement industry—cement, of course, formed the basis of concrete. For years during the time he had been associated with the industry, one of the principal bug-bears of the cement manufacturer had been the complaints he had received from time to time as to the failure of concrete. Nine times out of ten, upon investigation, these failures turned out to be due to dirty concrete aggregate being used. It would be a pleasure, in future, to tell the gentlemen who complained, how easy and how cheap it was to remove this dangerous clayey material from the materials which were being used as concrete aggregate.

Mr. Payne said he had been extremely interested in the paper. What he had to say was in the nature of an enquiry in order to learn a little bit more. At the present time his firm were getting their ballast out in the old-fashioned way by a scoop. They had the great advantage that when it

came out of the river it was washed and clean. They got the ballast out, some of it, in large size—mixed—and were very keenly interested in being able to break the material up to the size they wanted, somewhere about half an inch, in the most economical way. They had tried breakers, but the jaws had curled up so quickly that this method proved expensive. The author of the paper had warned them that they must expect the jaws to give out in a week; this made it a very expensive item. It would be interesting to hear if he had any experience of breakers where centrifugal methods with bolts or weights flung out were used for crushing flints or clinker. Those were the two materials that his (Mr. Payne's) firm were mostly concerned with

Mr. White said there was one economical point he would like to refer to with regard to the construction of the crushers. His people had dealings with crushers for coal, and they often experienced colliers' drills and other foreign matter getting in the rolls and sometimes breaking the gears. What they had done to remedy this was to put shear bolts in a flange on the shaft. If foreign matter got in, the bolts sheared, and the crusher rolls were saved.

The Lecturer said it was common to put cast-iron lugs on the rolls, so that these lugs broke off when steel got between them

Mr. White: You can do that safely when crushing rock.

The Lecturer: It is only done with rock.

Mr. S. Bylander said that in connection with this paper it would be a great advantage if the author could advise on the commercial practice of naming the various sizes. He had had a good deal of trouble during many years as to what was meant by half-inch gravel and what was meant by three-quarter inch gravel. He had taken up the custom of specifying that three-quarter inch crushed gravel was material which had passed through a screen with one inch diameter holes.

Mr. Sargeant, replying to the questions, said, as to the crushing of flints, he believed the Hockley crusher would do that work satisfactorily. There was a firm that made a very good crusher with hammers that flew out. As to the size of holes, some engineers specified that three-quarter inch matter should pass through three-quarter of an inch holes, but if an engineer specified that, he got not three-quarter inch, but five-eighths inch. Every piece of aggregate was jagged on the edges. For three-quarter inch aggregate inch holes would be too big: he should think that seven-eighths inch holes would pass three-quarter inch matter.

THE ANNUAL GENERAL MEETING.

The thirteenth annual general meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, London, S.W., on Thursday, April 27, 1922. Mr. E. Fiander Etchells (President) presided.

The minutes of the last preceding meeting having been taken as read, and confirmed, the Chairman presented the annual report and balance sheet. He said that report showed that the Institute was making good progress. It was in an increasingly sound position, and there was every evidence that this position would be still further improved and consolidated.

Major PETRIE (Chairman of the Finance and General Purposes Committee) moved the re-appointment of the auditors, Messrs. Monkhouse, Stoneham & Co.; and this was agreed to.

Mr. W. J. H. LEVERTON moved the adoption of the report which, he said, was an extremely satisfactory one. A point he would like to refer to was that the Chairman in his Presidential Address had referred to the training of the concretor, and had also spoken on the same matter in a lecture at the Building Exhibition. It was really a most important matter, and it would be wise for the Institute to take it up as quickly as possible, for some people fought rather shy of reinforced concrete, owing to doubts of the workmen whom they would have to employ.

The report was unanimously adopted

The CHAIRMAN, referring to the remarks of Mr. Leverton on the training of concretors, said that the Council was prepared to receive suggestions from any of its members on this matter.

Dealing with the subject of his re-election as President, Mr. ETCHELLS said he had been condemned to twelve months' hard labour, having been re-elected President. (Applause.) As this was the last general meeting of the session, he would take the opportunity to pass one or two remarks about the work of the staff. It would be remembered that the organising work was not all done by the Council. Some was done by the general body, but a lot was done by the staff; and the success of an Institution, he thought, depended more upon the Secretary than upon any other individual. In their own case the Secretary was very hard-working, and was an excellent organiser, and upon him, as much as upon anybody, would depend the success of the Institute. They must not forget, either, the Assist-

ant-Secretary, Mrs. Morris. She had been very hard working; the principal part of the work in connection with the examinations, which were an increasing success, had fallen upon her. Her tact and her well-balanced judgment were an asset which could be praised but not appraised. In the case of a staff, there was always some work, the value of which could not be estimated, and which could not be paid for in money, and he asked the meeting to give its thanks to the Secretary and Assistant-Secretary who both deserved them. (Applause.)

The announcement by the Chairman of the names of those who had been elected Members of the Council followed.

The medal of the Institute was then presented to Professor F. C. Lea for his paper on "The Modulus of Elasticity of Concrete."

The CHAIRMAN said that in making this presentation he would like to draw attention to the fact that, as the result of the strenuous work of preparing the paper, an eye trouble had attacked Professor Lea, who had been unable to see anything for three or four days; that is to say, he was temporarily blinded on account of voluntary scientific work done for his fellow men. The paper had been an admirable one. There had been a lot of work in it, and that work of a difficult kind.

Professor LEA was presented with the medal amid applause.

REVIEW.

"Reinforced Concrete."—A Practical Handbook for use in Design and Construction, by R. J. Harrington Hudson, B.Eng., A.M.Inst.C.E., A.M.I.Mech.E., M.Am.C.I., Member of the Science Committee of the Concrete Institute; with a foreword by E. Fiander Etchells, A.M.Inst.C.E., A.M.I.Mech.E., Hon. A.R.I.B.A., &c., President of the Concrete Institute, Chapman & Hall, 16/–.

No more fitting description can be found for this publication than that contained in the foreword, which cites the book as "one of the milestones on the high road of progress," and refers to its excellence as a text book and to the painstaking and masterly treatment of several special problems, with deduced formulæ which lie in the direct line of evolution of the Theory of Structures.

The special feature of the book is its analysis of the problems arising from considerations of the monolithic nature of reinforced concrete construction especially as regards the interdependence of beams and pillars, and the author's prefatory remarks draw attention to the necessity of stricter mathematical investigation in designing truely monolithic structures.

The foreword intimates that these problems are treated algebraically and semi-graphically by the not-too-intricate slope-deflection method, with special application of Clapeyron's theorem of three moments; and

Mr. Etchell's brief bibliography in this connection is in itself very instructive as is likewise his historical résumé in regard to the decimal separatrix, etc., with reference to the advantages of the Standard Notation of the Concrete Institute. This notation is observed throughout the text, and the book thereby gains much in utility and readability.

It is a far cry from the delicate mathematical location of the points of contra-flexure to considerations of the proportioning and mixing of materials, but the whole range of reinforced concrete construction is covered, and the earlier chapters give much reliable practical information on materials, and useful data as to the construction of forms, etc. In the latter connection a comprehensive perspective drawing on Plate I showing the typical arrangement of floor and pillar centering should prove especially valuable to the student, and to the engineer not altogether familiar with this branch of the subject.

The comment made in the chapter dealing with "weights, working stresses and other data," upon the real working factor of safety in reference to the elastic limit or yield point of the materials as opposed to the less rational "Factor of Safety of 4," based upon the "breaking" strength of the materials, introduces to the student and the general civil engineer an aspect of this important subject long entertained by the specialist structural engineer, and the plea in the same chapter for recognition in statutory regulations, of designers who take care to compute secondary stresses, by allowing such designers somewhat greater working stresses than those laid down at present, certainly voices an overdue concession.

The book contains a number of useful tables and graphs (whose usefulness would have been enhanced had they been printed and lithographed to a larger scale) giving, *inter alia*, simplified formulæ for the design of rectangular beams agreeable to two alternative stress proportions ; a table showing the effect of variation in the mixture of concrete upon the position of the neutral axis ; a set of graphs for the design of beams to various stress combinations, and a very complete table of Standard Tee Beams (Plate IV) containing nearly 1,500 references.

After dealing with the principles and practical design of beams and slabs in so far as the "flange" reinforcement is concerned, the author devotes a short chapter to "Shear and Diagonal Tension," which is supplemented by an article on "Shear" in the appendix.

As stated by the author himself, "opinions differ with regard to the relative value of diagonal rods compared with vertical stirrups," and generally, it may be said, that the matter of "Shear" reinforcement requires a deal of further investigation. What is needed is research to ascertain to what extent the usual design of reinforced concrete beam acts as a composite truss and to what extent it functions as a homogeneous beam in so far as the "shearing" stresses are concerned ; or in other words an answer is required to the question—How exactly is the increment of direct stress in the compression area and the tensile reinforcement developed ?

This section does not go much beyond a clear demonstration of the usually accepted principles, with practical formulæ for design, and it is not perhaps surprising that within the scope of "A Practical Handbook" the author has not benefited the reader with research in this direction as he has done in the matter of interdependent bending moments, although it may be submitted that the mutual flexure of monolithic members *of reinforced concrete* is to an appreciable extent governed by the distribution of "Shearing" strains.

Likewise the following chapter on pillars is not concerned with the development of any new semi-rational pillar formulæ (it is doubtful whether any entirely rational *practical* formulæ could be evolved) but the effects of slenderness, end conditions, etc., are referred to the values given in the L.C.C. regulations. Formulæ, however, for the combination

INDISPENSABLE BOOKS

Detailed Prospectuses C.I. on application.

CONCRETE: Plain and Reinforced. By FREDERICK W. TAYLOR, D.Sc., and S. E. THOMPSON, M.Am.Soc.C.E.

A treatise for practising engineers and contractors which may also be used as a text and reference book for engineering students. Covers the entire subject of concrete work, including processes, cement specifications, cement classifications, cement testing, proportioning, qualities of material, quantities of material, strength, mixing, laying, freezing, fire and rust protection, and waterproofing.

Third edition, revised and enlarged; 885 pages; 262 figures; many tables and diagrams. Price **30s.** net.

PRINCIPLES OF REINFORCED CONCRETE CONSTRUCTION. By F. E. TURNEAURE and R. E. MAURER.

This book presents in a systematic manner those principles in mechanics underlying the design of reinforced concrete. The results of all available tests that will aid in establishing coefficients in working stresses are given, and illustrative material from actual designs which will make clear the principles involved is also included.

485 pages; profusely illustrated. Price **20s.** net.

PLAIN AND REINFORCED CONCRETE ARCHES. By Prof. J. MELAN.

Gives a simple comprehensive treatment of stresses in reinforced concrete sections, and presents analytic and graphic methods for the complete design of all types of concrete arches.

161 pages; 43 figures and a folding plate. Price **12s. 6d.** net.

REINFORCED CONCRETE: A Practical Hand Book for Use in Design and Construction. By R. J. HARRINGTON HUDSON, B.Eng., M.C.I., with a Foreword by **E. FIANDER ETCHELLS,** A.R.I.B.A., President of the Concrete Institute.

Demy 8vo; 342 pages; 131 figures; numerous tables. Price **16s.** net.

THE THEORY AND DESIGN OF STRUCTURES. By EWART S. ANDREWS, B.Sc. Eng. (Lond.).

This volume contains 18 chapters on all phases of structural engineering.

637 pages; Demy 8vo; Illustrated. Price **13s. 6d.** net.

CONCRETE WORK. By W. K. HATT, C.E., Ph.D., and W. C. VOSS, B.S.

Deals extensively with the following subjects: Reading Blue Prints, Calculation of Quantities, Estimates of Cost, Selection and Inspection of Materials, Selection and Disposition of Equipment, Organisation of Gang, Staking Out, Excavating Form Work, Measuring, Mixing and Depositing Concrete and Protection of Work, etc.

451 pages; 224 figures; 20 full-page plates. Price **23s.** net.

CHAPMAN & HALL, LTD., 11, HENRIETTA ST., LONDON, W.C.2.

of direct and flexural stresses are clearly demonstrated and the chapter contains some very useful tables and graphs giving neutral axis ratios for varying eccentricities of loading on rectangular pillars with varying ratios of reinforcement.

The general body of the book concludes with tables, specifications and regulations (L.C.C.), and a very complete practical example worked through in extenso, of a floor carried by secondary and main beams supported upon pillars. This should prove very helpful to the student, but in considering the shearing stresses in the main beams it should be borne in mind that provision against the major shearing forces does not necessarily ensure sufficient provision for the local secondary shearing stresses evoked at the connections of the secondary beams.

Preceding the last but most important part of the book (namely, that devoted to secondary stresses in truly monolithic structures) is the standard notation of the Concrete Institute, up to date; and both the practising engineer and the student will find the 16 pages devoted to this, extremely useful, especially as all the International Ideographs, such as signs of variation, approximation, inequality, etc., etc., are included.

In the latter 80 pages of the book (including the appendices) chiefly devoted to investigations into bending moment values resulting from the interdependence of the flexure of beams and pillars, the author, as the result of great labour, has presented numerous very useful equations, qualifiers, diagrams and alignment charts for a comprehensive variety of combinations of loadings and groupings of spans and storeys, and these should be found a very valuable guide to the more correct design of monolithic structures.

In this connection it may be remarked that Professor F. G. Coker's photo-stress investigations upon rectangular homogeneous monolithic connections show that the stress distribution is very complex, and this condition may be still further aggravated when dealing with an heterogeneous material like reinforced concrete. It may be remembered also that Dr. Oscar Faber's text books comment upon the complicacy of stress in sharp bends.

The further assumption as to the perfect direction fixity of pillars at their bases will also need to be kept carefully in view, especially when dealing with "long" pillars in which an equivalent effect to initial curvature by misplacement of the longitudinal and/or lateral reinforcement, variation in joint efficiency of the reinforcement, etc., may readily partially nullify the theoretical effect of direction fixity; and although the flexure of the pillars has been investigated with reference to the major bending moments only, the secondary moments resulting from deflection eccentricity may appreciably modify the position of the points of contra-flexure in the pillars. Then again it is an unfortunate circumstance, as pointed out long since by M. Bressé in a similar connection, that whilst these investigations in so far as the beams are concerned are made with the chief object of proportioning the reinforcement economically to suit the varying bending moments, yet they are all based upon the assumption that the inertia moment of the beams is constant throughout.

The appendices give helpful notes upon the application of the calculus to the flexure of beams, and lead up to a demonstration of the theorem of three moments and the evolution of the typical formulæ upon which the valuable preceding particulars are based.

In conclusion the work may be commended as not only a comprehensive handbook, and a text book ranking with the works of Dr. Faber and others, but as one which should save the designer much arithmetical labour, give the specialist engineer a wealth of information and inspire the further research of the mathematician and physicist.—G. A. GARDNER, M.C.I., M.Math.Assoc.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *SEPTEMBER, 1922.* **No. 9.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

ANNUAL SUBSCRIPTION, 1922.

Article 13 "*. . . Subscriptions shall be payable in advance on the 1st day of January, when the financial year shall . . . commence.*"

MEMBERS	pay	First 500 ...	£1	1s.	0d.
		Remainder ...	£2	2s.	0d.
ASSOCIATES	,, ...		£2	2s.	0d.
ASSOCIATE-MEMBERS	,, ...		£1	11s.	6d.
GRADUATES, STUDENTS	,, ...			10s.	6d.

It is hoped that all Members (in any class) who have not adopted the Permanent Cheque method of payment will use the form for that purpose which may be obtained from the Secretary.

NOTICE.

In view of important resolutions which the Council intends placing before the Members very shortly, it is hoped that all Members whose subscriptions are not fully paid up will make a point of setting this matter in order, so that every Member may be in a position and qualified to vote. Particulars and statements of accounts may be obtained on application to the Secretary. (Telephone: Victoria 2112).

EXTRAORDINARY GENERAL MEETING.

An Extraordinary General Meeting of The Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, London, S.W.1, on Thursday, September 28, 1922, at 6 o'clock p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., A.M.I.Mech.E., Hon. A.R.I.B.A., &c.

The Secretary having read the notice convening the meeting, the President briefly addressed the members present on the subject of the meeting.

Professor HENRY ADAMS (Past President) proposed, and Mr. F. E. WENTWORTH-SHEILDS (Past President) seconded, a resolution: "That the name of the company be changed to 'THE INSTITUTION OF STRUCTURAL ENGINEERS.'"

After debate, Mr. C. F. MARSH (Vice-President) moved, and Mr. H. K. G. BAMBER (Member of Council) seconded an amendment, "That the title of the company be changed to "THE CONCRETE INSTITUTE AND INSTITUTION OF STRUCTURAL ENGINEERS."

After further debate the Chairman put the amendment to the meeting, and declared it to be defeated by 98 votes against to 12 votes in favour.

The following resolution, proposed and seconded as above, was then put to the meeting: "That the name of the company be changed to THE INSTITUTION OF STRUCTURAL ENGINEERS," and the Chairman declared the resolution to be carried by 110 (one hundred and ten) votes in favour to 13 (thirteen) votes against.

Mr. H. D. SEARLES-WOOD (Past President) proposed, and SIR HENRY TANNER (Past President) seconded a resolution:

"That Article 3 of the Memorandum of Association of the Company be altered by omitting the several objects numbered 1 to 25 both inclusive therein, and by substituting the following objects in lieu thereof, namely:—

"(a) To promote the science and art of Structural Engineering in any or all of its branches.

"(b) To enable Structural Engineers to meet and correspond and to publish or communicate information relative to Structural Engineering, including the constitution, properties, and use of materials.

"(c) Subject to the provisions of Section 19 of the Companies (Consolidation) Act, 1908, in any lawful manner

to acquire, hold, and/or dispose of for the purposes of the Institution real and personal property, and any rights or privileges necessary or convenient for the said purposes, and subject as aforesaid to exercise in respect of property of any nature, all the rights and powers of an absolute owner of property of that nature.

" (d) To do all such other lawful things as are incidental or conducive to the attainment of the above objects."

There being no amendments, the Chairman put this Resolution to the meeting and declared it to be carried *nem. con.*

After debate upon the proposed new Articles of Association, Mr. J. Ernest Franck (Vice-President) proposed, and Mr. R. J. Harrington Hudson seconded, a Resolution, " That the new Articles of Association already approved by this meeting, and for the purpose of identification subscribed by the Chairman thereof, be and the same are hereby approved, and that such Articles be and they are hereby adopted as the Articles of the Company in substitution for and to the exclusion of all the existing Articles thereof."

Mr. W. J. H. Leverton (Member of Council) moved, and Mr. G. C. Workman (Member of Council) seconded the following amendment:—" That in Article 54 of the proposed new Articles of Association, the Entrance Fee and Subscription of Members be £2 2s. respectively, and not £3 3s. as proposed."

The Chairman put the amendment to the meeting and declared it to be defeated by 107 votes against to 3 votes in favour.

The Chairman then put the original Resolution proposed and seconded as above, to the meeting, and declared it to be carried *nem. con.*

This concluded the business of the meeting, and the Chairman announced that the Resolutions carried would be submitted for confirmation to a further Extraordinary General Meeting to be held at the same address on Thursday, October 19, 1922, at 6 o'clock p.m.

PAPERS FOR SESSION 1922-23.

The Secretary of the Institute would be glad to receive from members offers of papers to be read during the forthcoming sessions. Offers should be made if possible before Monday, October 16th, and should be accompanied by a summary of the paper, for submission to the Literature Committee.

*THE EFFECT OF HUMIDITY, TEMPERATURE AND WATER CONTENT ON THE STRENGTH OF CONCRETE.

By ERNEST JOHN HAMLIN, D.Sc., A.M.Inst.C.E., etc.

INTRODUCTION.

The author has divided the following paper into two portions. The preliminary work was finished by the author in 1917, and is marked "Preliminary Researches." When the author returned to South Africa in 1919, he was fortunate to travel with W. J. Cooper, Esq., the Chief Engineer to the Penarth Cement Works, who brought several facts to the author's attention. Mr. Cooper discussed the use of concrete mixers with the author, and suggested that the water content was an important branch of research in South Africa. The author has tested over 3,000 specimens, and gives these results marked "Further Researches."

PRELIMINARY RESEARCHES.

Concrete is now being used extensively in many classes of constructional work, but in South Africa the atmospheric conditions are entirely different for different parts. A few miles distance often means a difference in altitude of 2,000 or 3,000 ft., etc., etc. The author, feeling that these different weather conditions, as shown in Figure 1, must have some direct bearing on the strength of concrete, has investigated, in a limited degree, how the strength varies under different temperatures, and in dry and humid atmospheres. This knowledge is of economic importance, because the contractor (or engineer) must know when to remove his shuttering, etc.

The greatest difficulty was experienced to represent with any degree of accuracy atmospheric conditions. However, after some experiments with gas thermostats, the author had his attention called to an excellent design of electrical thermostat described later.

The author made 8 such thermostats, and after calibration it was found that the amount of variation in temperature was approximately—1·25 Fahr. This the author considered was sufficiently accurate for the purpose.

*Paper read before the Concrete Institute, London, December 14th, 1921.

MATERIALS USED.

The following gives a brief outline of the class of material used throughout the experiments.

Sand.—Sand was taken from the Plankenberg River in the Division of Stellenbosch, South Africa. It was very thoroughly washed and then spread upon trays and sun-dried. Its specific gravity was approximately 2·70. The average percentage of voids was 34.

Stone.—The stone used was Paarl Granite. It was a fairly uniform sample and after being carefully washed was such that it would pass through a $\frac{1}{2}''$ screen but not through a $\frac{1}{4}''$ screen. Its specific gravity was 2·87 with an average density of 1·466. The average percentage of voids was approximately 48.

Cement.—Pretoria Portland cement was used throughout; the usual tests of the British Engineering Standards Committee as regards fineness and tensils were taken for each mixing.

The average values for the tensile strength obtained were :—

(a) For Pure Cement Briquettes—
556 lbs. per sq. in. at 7 days.
624 ,, ,, ,, ,, ,, 28 ,,

(b) For Briquettes using 3 Sand and 1 Cement—
178 lbs. per sq. in. at 7 days.
300 ,, ,, ,, ,, ,, 28 ,,

METHOD OF MIXING.

All the mixing was done by weight because in the author's opinion a more accurate result is obtained. In practice, it is usual to mix Concrete by volume, and so that the results would have some definite working value the mixture adopted was that represented by the much used: 1 Cement, 2 Sand, and 4 Stone. By weight the following are the proportions required : 1 Cement, 1·77 Sand, and 4·51 Stone.

The amount of water used was slightly different for different temperatures of test and the following were used : 10·25% for temperature 55° Fahr., 10·5% for temperature 70° Fahr., 10·75% for temperature 85° Fahr., and 11% for temperature 100° Fahr. All these percentages are of the total value of the constituent Dry Materials.

By using the above method of mixing a very accurate "mix" ought to be obtained and the author was disappointed that the results recorded were not more consistent.

Six specimens were made and tested, but only the four highest values were tabulated.

The specimens were very carefully mixed on a steel slab and thoroughly worked with a small steel trowel. The cement and sand were weighed out into small subdivisions of the total weight, and each subdivision very carefully mixed, so that when all these small subdivisions of sand and cement were gathered together, the whole mass presented a uniform appearance.

The stone was then added, and the whole of the constituent dry materials were given six complete turnings. This gave an excellent mixture.

The necessary weight of water was then added, and the whole thoroughly mixed into a compact mass.

The time of mixing was kept approximately constant, and the various briquettes were moulded in a steel former to the size of $4\frac{1}{2}$ in. by $4\frac{1}{2}$ in. by $4\frac{1}{2}$ in.

TEMPERATURE ADOPTED.

Only four temperatures were used in the test, namely, 55° Fahr., 70° Fahr., 85° Fahr., and 100° Fahr. These temperatures are usual working temperatures at different seasons in South Africa.

CONDITIONS OF ATMOSPHERE IN THERMOSTAT.

To keep the atmosphere in the thermostat as dry as possible, flat trays (perforated) containing calcium chloride were placed on special shelves in the thermostat whilst the saturated —or humid—condition was obtained by filling similar trays with water.

AGE OF TEST SPECIMENS.

Six different ages of specimens were used, namely :— 3 days, 7 days, 10 days, 14 days, 21 days and 28 days. The results obtained are shown in Tables I, II, III and IV.

RESULTS.

Figure 2 shows how the strength of Concrete :—

(A) Increases with age.

(B) Increases with temperature during setting periods.

(C) Increases if atmosphere is kept humid.

Figure 3 shows more clearly how the humidity of atmosphere affects the strength of concrete. The ratio of strength "dry" to the strength "humid" is plotted against the age in days. At 100 Fahr. it will be noticed that the concrete which has aged in dry atmosphere is 6·2% weaker than that which has aged in humid atmosphere.

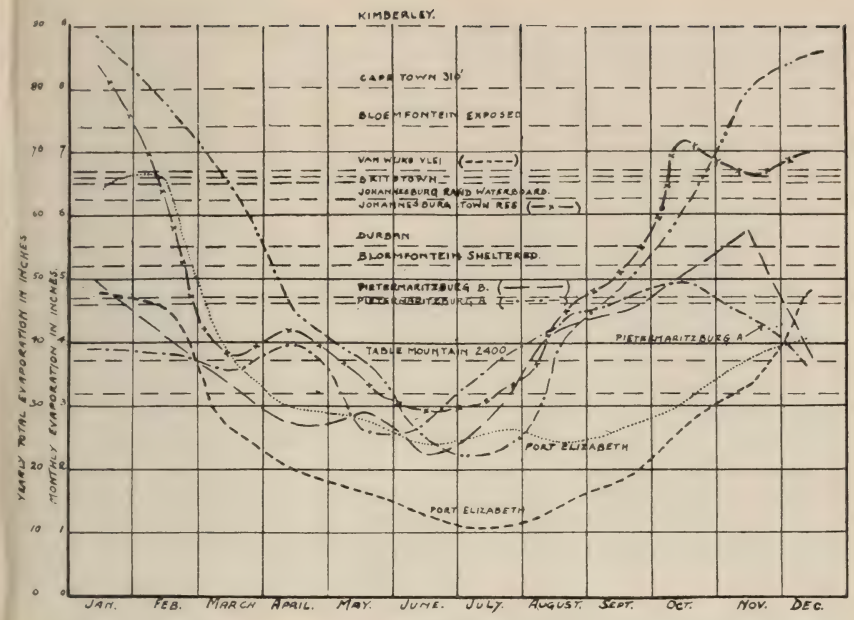


FIG Nº 1.

CURVES SHEWING HOW WEATHER CONDITIONS VARY WITH THE TIME OF YEAR AT CERTAIN PLAC

CURVES SHEWING HOW CRUSHING STRENGTH VARIES WITH AGE.

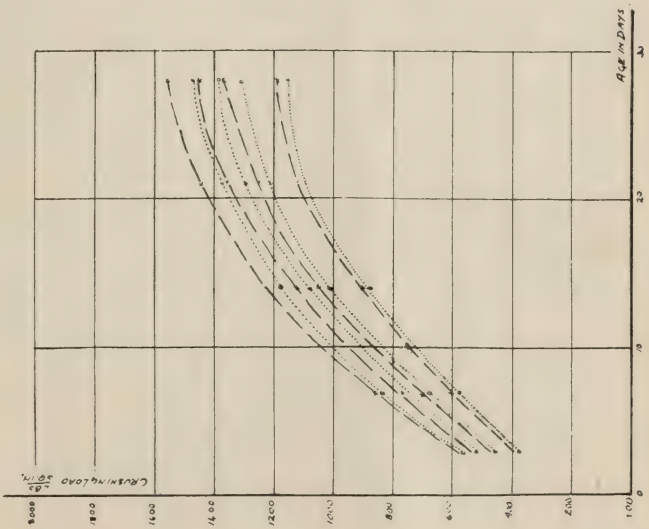


FIG. 2.

In Figure 4 curves have been drawn to show how the strength varies with the temperature. From the results and these curves it is fairly easy to obtain the strength at any time during the working limits of temperature.

CONCLUSIONS.

The author believes that the following conclusions can be stated definitely.

1. Under uniform temperature conditions there is a general increase in strength during the first 28 days of ageing.

2. Concrete maintained at a temperature of 70° Fahr. will have a very much greater strength in a week "after mix" than concrete at a temperature of 40° Fahr.

3. Figure 3 seems to indicate that the strength of concrete decreases very rapidly if allowed to age in a "dry" atmosphere. The author is convinced that dis-integration would take place if the time were increased.

4. It is imperative in Tropical and Sub-Tropical Countries to take account of atmospheric conditions, especially in such work as concrete water-towers, concrete piers for bridges, etc., etc.

FURTHER RESEARCHES.

As stated above the author was able to have the advice of Mr. Cooper in 1919. The author felt sure at that time that the use of concrete mixers in some cases had been the cause of the failure of concrete put into structures. A more or less sloppy mixture is necessary for the successful use of mixers.

After a great deal of discussion, such as would only be possible under the conditions pertaining at that time—namely 30 days at sea with a concrete expert— the author concluded that there must be a definite quantity of water to be used in order to obtain economically the best mixture.

Bearing in mind the great range of temperature and humidity in South Africa, the author, on the advice of the above-mentioned gentleman, continued his experiments in concrete to include these two factors as well as water content.

From a paper read by Mr. R. K. Skelton before the Connecticut Society of Engineers in 1915, it appeared that the critical water content under the conditions he experimented was 27·5% by weight of the quantity of *cement* used.

The author felt that humidity would alter this figure and so conducted his experiments giving the range of water content between 20% to 30% by weight of the quantity of cement used.

MATERIALS.

The materials used in these experiments were exactly the same as those for the "Preliminary Experiments."

METHOD OF MIXING.

The same method was adopted as before, namely by weight, but was arranged that the mixture would be equivalent to a mixture of: 4 stone, 2 sand and 1 cement by volume.

The forms used were the same throughout, and the mixing was done on lines previously adopted by the author.

In addition to the quantity of water used, as a definite percentage of the weight of cement used, an additional amount was added to compensate for the absorption of the water by the stone and sand.

The Paarl Granite required 2·04 per cent. of its weight, and the Plankenberg River Sand required 9·42 per cent. of its weight.

TEMPERATURES ADOPTED.

These were the same as those used previously, but the number of thermostats used had to be increased to 18.

AGE OF SPECIMENS.

The same six different ages of specimens were used.

TABULATION OF RESULTS.

There were six specimens prepared and tested for each percentage of water used, for each age of specimen, for each temperature, and for each condition of humidity, making a total of 3,128 specimens tested.

For each condition the four highest readings were taken and results tabulated in Tables 5, 6, 7 and 8, and plotted in Figures 9, 10, 11, 12 and 13.

CONCLUSIONS FROM CURVES.

(1) The curves bear out definitely the author's conclusions arrived at by the preliminary experiments.

(2) That there is a definite quantity of water which gives the maximum strength of concrete.

(3) That this quantity depends on the humidity and temperature of the atmosphere.

(4) That in using concrete mixers contractors must be careful of the quantity of water used, and consulting engineers should specify this quantity.

(5) That consulting engineers should examine the stone and sand before writing the specification for concrete.

CURVES SHEWING HOW THE RATIO STRENGTH 'DRY' TO STRENGTH 'HUMID' VARIES WITH THE AGE

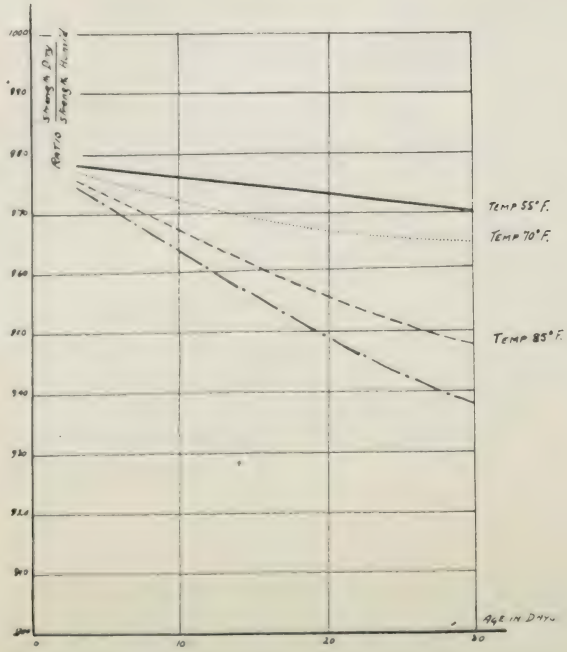


Fig. 3.

CURVE SHEWING HOW CRUSHING LOAD VARIES WITH TEMPERATURE & HUMIDITY.

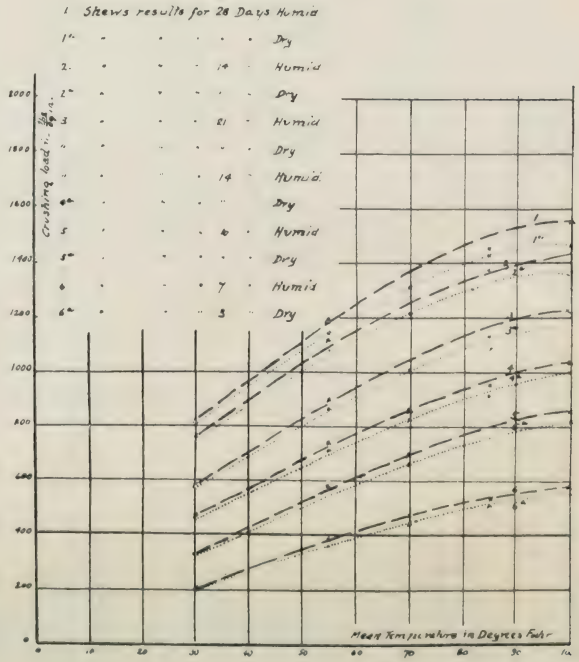


Fig. 4.

TEMPERATURE 55°

CURVE SHEWING HOW CRUSHING STRENGTH VARIES WITH WATER CONTENT.

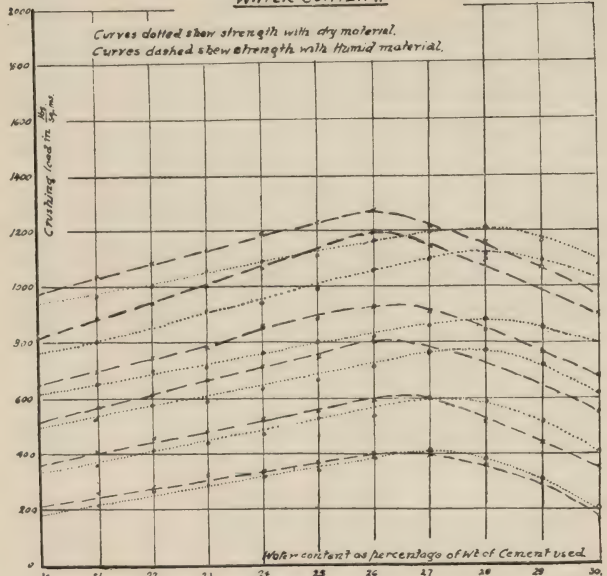

Curves dotted shew strength with dry material.
Curves dashed shew strength with Humid material.
Crushing load in lbs. per sq. inch.
Water content as percentage of Wt of Cement used.

FIG. 9.

TEMPERATURE 70°

CURVE SHEWING HOW CRUSHING STRENGTH VARIES WITH WATER CONTENT

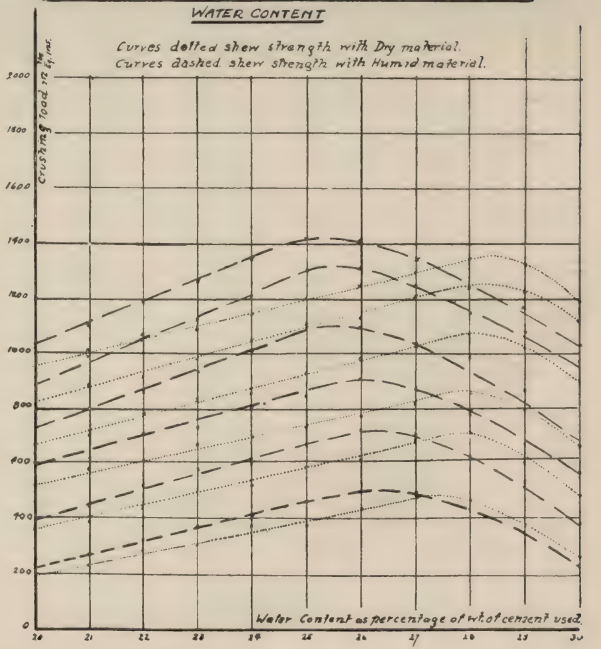

Curves dotted shew strength with Dry material.
Curves dashed shew strength with Humid material.
Crushing load in lbs. per sq. inch.
Water Content as percentage of what cement used.

FIG. 10.

TEMPERATURE 85°.

CURVE SHEWING HOW CRUSHING STRENGTH VARIES WITH WATER CONTENT.

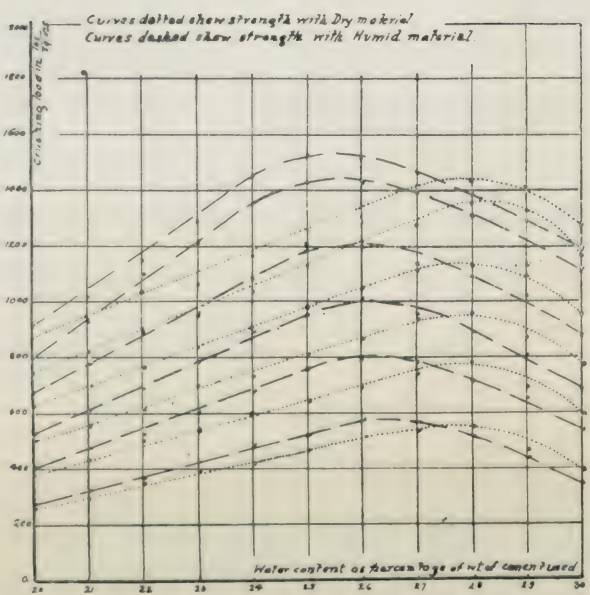


FIG. 11.

TEMPERATURE 100°.

CURVE SHEWING HOW CRUSHING STRENGTH VARIES WITH WATER CONTENT.

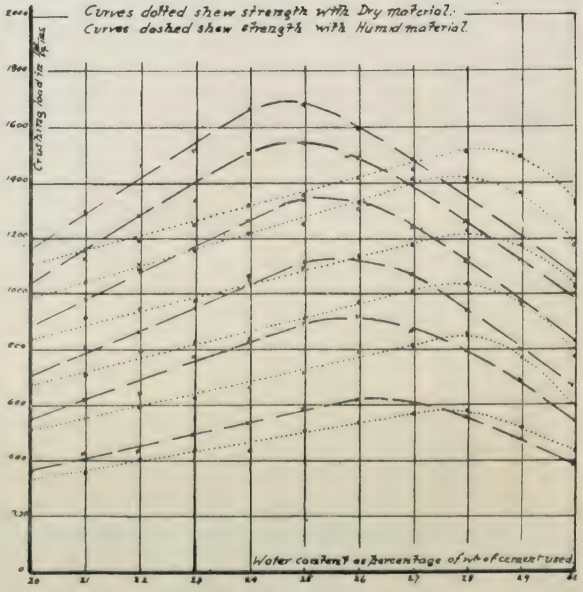


FIG. 12.

(6) That it is essential that the average humidity and temperature of the position where the concrete is to be fixed should be known.

NEED FOR FURTHER RESEARCH.

The author hopes to continue these experiments for varying mixtures of aggregate and cement and for different kinds of aggregate and sand; for the author feels that cement concrete must in the future be far more scientifically handled than it has been in the past.

CURVES PLOTTED FROM FIGS. 9.10.11.12.
SHEWING HOW MAXIMUM STRENGTH VARIES WITH WATER CONTENT AND TEMPERATURE

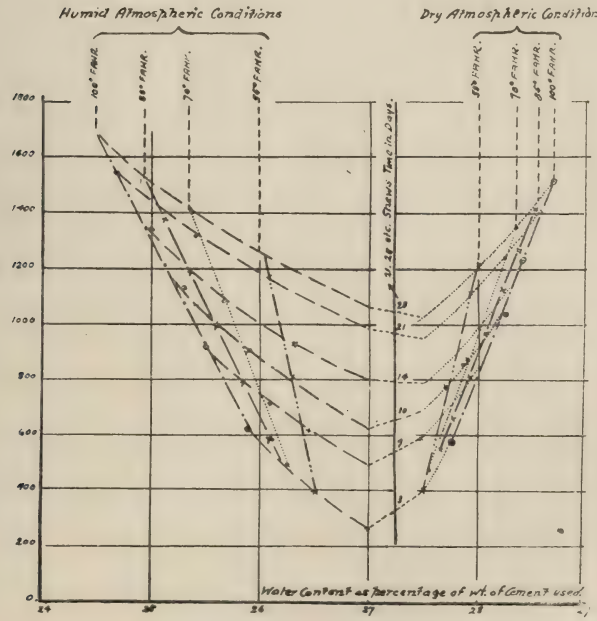


FIG. 13.

NOTE.—Owing to pressure on space it has been found impossible to include the other diagrams prepared by the Author. These may, however, be seen at the offices of the Institute.

DISCUSSION.

The Chairman announced that Mr. Hamlin was prepared to answer any questions on his paper.

MR. COLSON: You spoke of the aggregate as being granite. Have you any idea as to the amount of water which that granite took up? It must make a great difference in the percentage of humidity in the whole mixture.

MR. HAMLIN: The Paarl granite absorbed 2·04 times its weight, and the sand absorbed 9·42 times its weight of moisture.

DR. OWENS: I should like to ask, sir, if I might, if Mr. Hamlin has any information on the relative weights of the specimens which were tested in humid and dry air? Apparently the only possible difference could have been due to difference in water contents resulting from either the humidity or the effect of the dry air, and that should be brought out by weight. If he has not got those data, I suggest that it would be a very useful addition in further tests. One other point. The fact that you place a specimen in dry air does not necessarily mean that it is subjected to the greatest possible evaporation, as one of the most important factors covering evaporation is wind, the movement of the air. In fact, if the specimens were placed in still air, I do not think they would be subjected to the greatest possible evaporation, and it is possible that placed in less dry air with more movement he would have got quite different results. The importance to me seems to be this, that it is not quite justifiable to assume that the curves on the dry side represent the conditions which would be got under all circumstances of drying.

MR. W. A. GREEN: We shall probably have many more questions to ask after we have had the opportunity of studying this very interesting paper in more detail. There is one question. I am not quite clear if in his proportions by weight, the sand and the granite were dried before they were weighed. Were they absolutely dry? He spoke of the water content being taken after the materials were soaked, but were they dried first?

PROFESSOR MATTHEWS: I should like to say how much I have appreciated the paper and while I have not studied the action and the effect of humidity on concrete, I have studied a subject which is very similar, or rather on the same lines; that is the action of excessive heat and the action of excessive cold on concrete. I think we have not given sufficient attention to these important things, and, as far as I can

recollect now, for the benefit of the members, I will just in a word say what the effect of excessive cold was. I carried out a number of experiments lasting two years, and my conclusions were these: Light frost, not exceeding 3·7° of frost does not affect freshly mixed concrete, but a temperature of 17° of frost does seriously affect the concrete; so much so that it reduces its strength considerably. Concrete under those conditions gave a much less favourable result to compression and tension than in the ordinary way. I thought the members would like to know that 3·7 degrees of frost, continued for half an hour, does not affect the concrete, but 17 degrees of frost seriously cripples it; and it can never recover.

THE CHAIRMAN: Does Professor Matthews refer to concrete which has been well matured but is containing some water?

PROFESSOR MATTHEWS: I refer to freshly mixed—in a plastic state when the test is made. Only just mixed the minute before.

MR. HAMLIN: Seventeen degrees lasting only for half a minute?

PROFESSOR MATTHEWS: No, 17 degrees continuously for several hours. All that time the concrete is getting more and more set, and is able, one would have thought to resist the cold better. Seventeen degrees of frost cripples the concrete. I was thinking, Mr. Chairman, that if the reinforced concrete structures are designed in England for some country across the sea, somewhere where they are subject to severe frost, it is quite easy for the engineer to have a great difficulty in knowing what amount of water to mix with the concrete, and how to arrange so that no mixing is done when the temperature is below a certain point.

THE CHAIRMAN: Has Professor Matthews carried out any tests in cases where the concrete was mixed at about freezing temperature, but afterwards a lower temperature occurred. For example, workmen may carry out work at say four or five o'clock on a winter's afternoon, when the temperature is about freezing. During the night a heavy frost sets in and the temperature falls so that at 3 a.m. it is many degrees below freezing.

PROFESSOR MATTHEWS: Three hours, I think, was the range. I think that after a three hours set it is able to resist the cold. But the freshly mixed only just come from the mixing machine, or mixed by hand, gave the result I named.

Mr. S. BYLANDER said that some information might be interesting on the question of concrete setting in wet weather. He had had the supervision of concrete work in Canada at temperatures varying from 80 degrees above to 30 below zero. He had noticed that in hot weather, say at 80, 90, or 95 degrees, the concrete had dried out very quickly; and it became necessary to protect the concrete from the hot wind. Where the concrete was not so protected he had noticed that the surface, say a quarter of an inch on the top, had dried off very quickly and would not harden. With regard to concrete placed in wet weather, he had made a number of tests to ascertain what was the real effect of freezing the concrete. He had had a series of nine cubes, three in a set, and had subjected these to various conditions. The temperature of the air when the concrete was placed had been 50 degrees below zero. One set of three cubes had been placed on the wall and subjected to the wind and the winter weather for about four months. It had been tested in the following June, after it had been frozen for about four months, had had the opportunity of setting off and then been thawed up. That concrete had shown a strength of about half what it would have had if properly matured. Other cubes had been placed in a warm room, and the effect of freezing only for an hour or two ascertained. The effect seemed to be that if the concrete was frozen immediately, thawed up and let freeze again, the result was very bad. Concrete which had been properly protected, particularly in great masses, and left for at least twelve hours to set before it froze, would attain a set practically the same as if properly matured. But one precaution must be taken; the gravel and the sand and other materials must be heated before mixing the concrete, and the structure must be strutted up and supported during the winter months until the concrete had time to set during the early summer.

Professor MATTHEWS said, with regard to heating the aggregate and so on, that he had found blocks made with hot water, and where the materials were heated, gave a less satisfactory result than those made with cold water and materials in the ordinary way.

Mr. COLSON said that to his knowledge concrete pipes were frequently made in moulds, taken out of the moulds as soon as they could be handled, put into heated chambers and heated there. He did not mean that the materials with which the concrete was made were heated, or that hot water was used; but the concrete itself after being made was heated, and, as they

knew, it set very hard. Of course this simply followed on the ordinary cement testing process and did not go against what Professor Matthews had said.

Mr. WHITE said he had been in Winnipeg when several thousand yards of concrete had been poured under very severe winter conditions; and it had been found that concrete made during the winter, if properly safeguarded, was quite serviceable. The method adopted in big work had been to cover with a tarpaulin and set coke fires below. It had also been thought that the concrete, by its chemical action, would generate a heat of its own; which would rather take away the effect of the frost. There had been one or two structures in Winnipeg badly constructed during the winter season. From these the boards had been taken away during the winter season, and when the spring came along and the thaw set in, those wonderfully strong looking jobs had gradually melted away.

Mr. HAMLIN said he believed the first question raised had been that of Dr. Owens as to the net weight of the samples tested. He (Mr. Hamlin) had stated in his paper that the concrete was composed of one part cement, two sand and four stone, that the specific gravity of the materials had been taken and that they had been weighed out quite dry for the set of tests he had shown that evening. It had then been found exactly how much moisture the sand took up and how much the stone; and afterwards the whole thing had been mixed together. He had not recorded the weights in all cases, but the weights of those he had tested before putting into the thermostat had not differed more than by about three per cent. when going into the thermostat for the first time.

Dr. OWENS: I think you misunderstood. I referred to the weights at the time of breaking.

Mr. HAMLIN said he has not recorded that. He thanked Dr. Owens and other members for hints they had given him, which would be something to work upon when he got back to South Africa. With regard to the question of evaporation, he quite recognised the importance of wind; but this was more important in England than in South Africa. In the Karoo there was usually no wind, and in Natal evaporation was more by heat than by wind. He believed that by an alteration in the thermostat he would be able to get a certain velocity of wind passing through. He hoped at same future date he might be able to give the Institute the result.

Mr. GREEN said his question had been whether the weights given were those of the materials dried.

MR. HAMLIN replied that the materials were dry before they took up the moisture, and the quantity of moisture they took up was recorded in the paper. The question of the effect of freezing he had not gone into. As to pipes being put into heating chambers, he had seen that done; but an experience of his own in that direction had been rather unfortunate. It seemed to him, as Mr. White had said, that the heat generated by the concrete itself would have some bearing on the matter.

REVIEW.

Building Supervision. By George W. Grey, P.A.S.I., Lic.R.I.B.A., M.C.I. 2nd edition. Small 8vo., pp. 176. London, E. & F. Spon. 1922.

This little book is thoroughly practical, and should be in the hands of every architect and clerk of works. The author has his knowledge first hand, and he does not fail to show up the scamping tricks that one finds from time to time among the lower order of building contractors. Their point of view regarding both labour and material, is not, "Is this up to specification quality?" but "Will this pass without being condemned?" The author's remarks on cement, sand, concrete aggregates and mixing are useful. He points out that broken Fletton bricks should not be used for aggregate as several failures have been traced directly to this cause. He discusses the question of a dry or wet mixture, and concludes with the majority of engineers that it should be dry enough to just bring the water to the surface on tamping, neither more nor less. He makes some useful remarks on the supervision of reinforced concrete work. He says also, "Close supervision of reinforced concrete is considered so necessary, that it has been suggested that clerks of works, competent to supervise this work, should be registered by the Local Authorities, and their employment made compulsory, where this material is being used." This is a matter the Concrete Section of the Institution of Structural Engineers should take into consideration at an early date and issue licences after a proper investigation into each case.

The heading of Chapter XI., "False Work," is a misnomer, there should be no false work about a building, the proper term is "Form Work." All the trades engaged on a building, as well as the materials employed, come under review by the author, and what he has to say should be thoroughly absorbed by the persons for whom he writes. If one may use the expression the book can best be described as an "eye-opener."—H. A.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *OCTOBER, 1922.* **No. 10.**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

NOTICES.

The proposition to change the title of the Institute has been duly passed and confirmed, all the resolutions at the Confirmatory Meeting being passed *nem. con.* A letter from the President dealing with the changes which have been made accompanies this copy of the *Journal*. The changes await the formal sanction of the Court, and until this is given they are not actually "official." It is hoped that this sanction will be received before the New Year.

The new Articles of Association have, however, become operative as and from October 19th, 1922.

ORDINARY GENERAL MEETING IN DECEMBER.

An Ordinary General Meeting will be held at Denison House, 296, Vauxhall Bridge Road, S.W. 1, on Thursday, December 14th, 1922, at 7.30 p.m., when a paper on "The Strength of Rectangular Slabs," will be read by Mr. A. Ingersleb, B.Sc. The paper will be illustrated by lantern slides.

The Conference for Securing Uniformity of Practice

in connection with the Sections of the L.C.C. (General Powers) Act, 1909, that deal with the Regulation of

STEEL FRAME BUILDINGS.

Chairman: Frederick R. Farrow, F.R.I.B.A., representing the Royal Institute of British Architects.

Vice-Chairman: Bernard J. Dicksee, F.R.I.B.A., District Surveyor for Southwark, representing the District Surveyor's Association.

W. E. Vernon Crompton, F.R.I.B.A. F. N. Jackson, M.Inst.C.E., Hon.A.R.I.B.A.	Representing the Royal Institute of British Architects.
W. R. Davidge, F.S.I., Assoc. M.Inst.C.E., A.R.I.B.A., formerly District Surveyor for Lewisham A. A. Fillary, District Surveyor for Greenwich Succeeded by A. Ashbridge, F.R.I.B.A., formerly District Surveyor for St. Marylebone E. A. Young, A.R.I.B.A., District Surveyor for East Lewisham and Woolwich	Representing the District Surveyors' Association.
S. Bylander (Deputy: Dr. Oscar Faber, O.B.E., D.Sc., Assoc.M.Inst.C.E., &c.) E. Fiander Etchells, Assoc. M.Inst.C.E., A.M.I.Mech.E., Hon.A.R.I.B.A., &c. W. G. Perkins, District Surveyor for Holborn	Representing the Concrete Institute.

Hon. Secretary: H. Kempton Dyson, formerly Secretary of the Concrete Institute.

SKELETON FRAME BUILDINGS.

REPORT OF THE CONFERENCE OF REPRESENTATIVES OF THE ROYAL INSTITUTE OF BRITISH ARCHITECTS, THE DISTRICT SURVEYORS' ASSOCIATION AND THE CONCRETE INSTITUTE, CONVENED FOR THE PURPOSE OF SECURING UNIFORMITY OF PRACTICE IN CONNECTION WITH THE SECTIONS OF THE L.C.C. (GENERAL POWERS) ACT, 1909, that deal with the regulations of SKELETON FRAME BUILDINGS.

SEVERAL DIFFICULTIES having arisen in the interpretation of the Act, and diverse opinions having been expressed upon the meaning of some of its requirements the District Surveyors' Association convened a conference of the Royal Institute of British Architects, the District Surveyors' Association and the Concrete Institute with the object of securing uniformity of opinion and practice in the application and administration of the Act, and of devising a working arrangement. Three representatives of each society were appointed:—

Representing the Royal Institute of British Architects:—Messrs. F. R. Farrow, W. E. Vernon Crompton and F. N. Jackson. Representing the District Surveyors' Association:—Messrs. Bernard Dicksee, W. R. Davidge and A. A. Fillary. Representing the Concrete Institute:—Messrs. S. Bylander, E. Fiander Etchells and W. G. Perkins. Subsequently Messrs. Davidge and Fillary were compelled, owing to other calls on their time, to resign, and Messrs. A. Ashbridge and E. A. Young were appointed to succeed them. During Mr. Bylander's absence abroad Dr. Oscar Faber acted in his stead. Mr. Farrow was elected Chairman, and Mr. Dicksee, Vice-Chairman. The conference held its first meeting on June 8, 1915, and 42 meetings in all have been held.

The Conference availed itself of the opinions of consulting and structural engineers and others; and, after carefully considering the various aspects of the subject, recom-

mend that the accompanying working methods be generally adopted.

Signed on behalf of the Conference:

FREDERICK R. FARROW, Chairman.
BERNARD DICKSEE, Vice-Chairman.
H. KEMPTON DYSON, Hon. Sec.

Note.—It is understood that these recommendations are put forward for guidance only, and that they do not impose any statutory obligation.

10th December, 1917.

CALCULATIONS GENERALLY.

All calculations may be made within a standard of approximation of 5 per cent.

The allowance to be made in ascertaining the equivalent dead lead due to live loads from lifts, hoists and other like moving loads should be taken as three times the weight of the cage and load.

The allowance to be made for rolling loads should be 50 per cent. increase on the actual loads; and for other loads that might be suddenly applied to a floor, without impact, an increase of 100 per cent.

GENERALLY AS TO PILLARS.

In determining the strength of pillars, the flange plates shall be assumed not to project beyond the outermost line of rivet centres more than

(a) Eight times the thickness of the thinnest plate in the flange, or
(b) $4\frac{1}{2}$ inches.

whichever may be the lesser. All metal beyond those limits shall be ignored in the calculations.

BEAMS.

In cases where the distributed load on a beam is calculated on a distance less than the full span, the bending moment shall not be taken at $\frac{Wl}{8}$ but shall be calculated for the exact disposition of the load

DIMENSIONS.

Dimensions for calculating the strength of pillars and beams should be taken generally from floor level to floor level for heights, and from centre to centre of supports for lengths on plan.

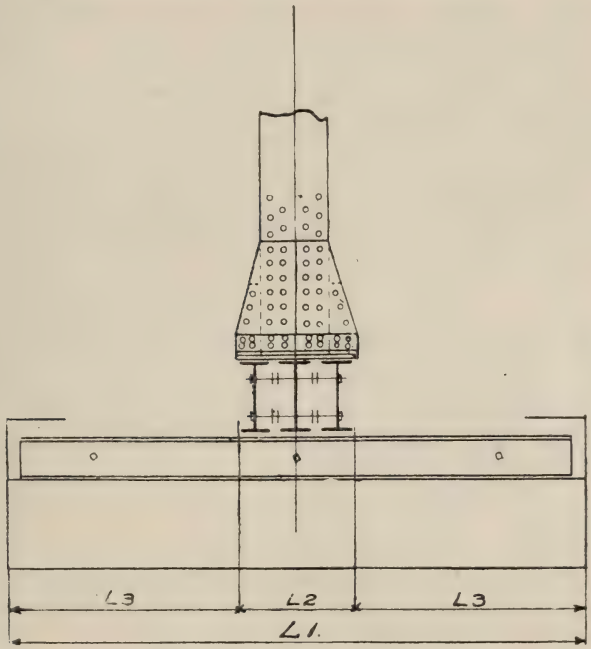
Where the ends of a beam are supported by a brick or masonry wall, the centre of support above referred to shall be taken as situated within the wall to the following extent from the internal face:—

(a) In the case of main joists or main girders—6 inches.

(b) In the case of filler joists or subsidiary joists—3 inches.

GRILLAGES

If the projection of the joists in grillages do not exceed three times the depth of the particular section of joists employed, no calculation for bending moment need be made.



If it exceed such projection, calculation may be made by the following equations:—

$$B_c = \frac{P}{8}\left(L_1 - L_2\right)$$

$$M = \frac{P}{5}\left(L_1 - L_2\right) = \frac{2}{5}\,P \cdot L_3$$

Where:

B_c = Bending Moment at centre in inch-tons.

M = Modulus of Section of entire grillage in inches^3.

P = Total pressure or load in tons.

L_1, L_2, and L_3 are in feet, measured as shown in diagram.

Where joists in grillages are spaced apart a distance at least equal to the width of the flange, both top and bottom flanges may be reckoned as giving bearing area.

ALLOWANCE FOR WEIGHTS.

The weights of the filler joists may be allowed for in the dead weight of the floor; but, in the case of main joists, the dead weight of the joists should be added.

For the purpose of estimating the total load to be carried, the dead weight of main joists may be covered by a general allowance of half a hundredweight per foot run for all sections up to and including 15 in. by 6 in. R. S. J.

The weight of any concrete encasing to main girders should be added to the dead load on the basis of one pound per square inch of cross sectional area per foot run.

STRENGTH OF STEEL BEAMS AND GIRDERS.

The section modulus necessary for mild steel beams of any type (including rolled steel joists) with ends freely supported, and loads uniformly distributed, is obtainable from the following equation:—

$$M = W \cdot L/5$$

Where:

M = Section *modulus* in inches ^{3}.

W = Total *weight* in tons, including weight of beam.

L = Effective *length* in feet between centres of reactions.

CALCULATION OF DEFLECTION.

When necessary, the deflection of steel joists and girders of uniform depth may be obtained from the following equation:—

$$dn = C \cdot W \cdot l^3 / E \cdot I$$

Where:
C = a *constant* (see following table).
dn = Total *deflection* in inches.
E = *Elastic* modulus in tons/inch^2.
I = Maximum moment of *inertia* of section in inches^4.
W = Total *weight* or working load in tons.

Method of Loading.	Value of C.	
	Uniform Section.	Uniform Flange Stress.
CANTILEVER		
Distributed Load	$\frac{1}{8}$	$\frac{1}{4}$
Concentrated Load at end	$\frac{1}{3}$	$\frac{1}{2}$
BEAM		
Distributed Load		
Ends of beam freely supported	$\frac{5}{384}$	$\frac{6}{384}$
Ends of beam partially fixed, say	$\frac{3}{384}$	say $\frac{4}{384}$
Ends of beam continuous over supports say	$\frac{2}{384}$	say $\frac{3}{384}$
Ends of beam fixed	$\frac{1}{384}$	$\frac{2\frac{1}{4}}{384}$ or $\frac{3}{512}$
Concentrated Load at centre		
Ends of beam freely supported	$\frac{8}{384}$	$\frac{12}{384}$
Ends of beam partially fixed, say	$\frac{6}{384}$	$\frac{9}{384}$
Ends of beam continuous over supports say	$\frac{4}{384}$	say $\frac{6}{384}$
Ends of beams fixed	$\frac{2}{384}$	$\frac{3}{384}$

A beam shall be deemed to have fixed ends when its ends are sufficiently secured to other parts of the construction having such rigidity as will maintain the neutral planes of the beam at the ends in their original positions and directions under all variations in the incidence and intensity of loading.

For the purpose of calculations the elastic modulus of steel in tension and compression shall be taken as follows:—

For joists and other single rolled sections:

$E = 13{,}400$ tons/inch^2.

For compound girders, plate girders and other riveted sections:

$E = 12{,}000$ tons/inch^2.

In order that beams and joists with ends freely supported and loads uniformly distributed may not be deflected beyond the limit allowed in Section 22 (7) the moment of inertia of any such beam or joist must be at least equal to the following:—

Filler joists embedded for their full depth in brick or stone concrete:

When the ratio of effective span to depth exceeds 28, then

I must be at least equal to $\frac{WL^2}{24}$

Main Joists or Beams:

When the ratio of effective span to depth exceeds 24, then

I must be at least equal to $\frac{WL^2}{18}$

Compound Girders and Plate Girders and other built up sections:

When the ratio of effective span to depth exceeds 20, then

I must be at least equal to $\frac{WL^2}{16}$

Where:

I = Moment of Inertia in inches^4

L = Length of effective span in feet.

W = Total weight or load in tons.

SPACING OF FILLER JOISTS.

The maximum safe spacing of filler joists depends upon the thickness and quality of the concrete, but the spacing that will stress a given filler joist up to 7.5 tons per square inch on the extreme fibre is derivable from the following equation:—

$$P = \frac{5 \cdot M}{W \cdot L^2}$$

Where:

P = Pitch or spacing in feet (centre to centre).

M = Section modulus in inches^3.

L = Length between centres of reactions in feet.

W = Intensity of total weight or working loads in tons per square foot.

CRANKED GIRDERS.

The determination of stresses in cranked girders is dealt with in Appendix I. to this report, and mansard roofs designed in accordance with the principles enunciated would be acceptable.

If the ends of cranked girders be free to slide, the reactions will be vertical; and the stresses are determinable by ordinary statical methods.

When the ends are prevented from sliding, and are thus fixed in position (though not in direction) there will be horizontal reactions in addition to the vertical reactions. These horizontal forces may be determined approximately by the following formulæ:—

For a fixed point load on top member:—

$$H = \frac{P}{2h}\left(\frac{2km+3(m+n)(l-2m)-3n^2}{2k+3(l-2m)}\right)$$

For a distributed load on top member:—

$$H = \frac{P}{12h}\left(\frac{4mk+(l-2m)(l+4m)}{\frac{2}{3}k+(l-2m)}\right)$$

For a distributed load on inclined leg:—

$$H = \frac{Pm}{24h}\left(\frac{5k+6(l-2m)}{\frac{2}{3}k+(l-2m)}\right)$$

The symbols are detailed in the Appendix, and shown on diagrams.

The total horizontal force will be the sum of the forces resulting from each load considered separately.

The vertical reactions are found as usual.

The total horizontal force and vertical forces once having been derived, the bending moments and thrusts are determined in the ordinary way.

The foregoing formulæ are, however, only applicable under special conditions, particularly as to maintenance against sliding, and care should be taken to see that the case in question is sufficiently in accordance therewith to warrant the use of the approximate formulæ; otherwise, if reliable solution be aimed at, more detailed investigation will become imperative.

RIVETING.—Sections 21, 22 (12).

(See Appendices II. and III.)

In order that the term " properly riveted " may be considered to be complied with, the following requirements shall be fulfilled:—

(1) The grip length between heads shall not exceed four times the diameter of the rivet, unless the calculation of the stresses include the stress due to initial tension due to cooling, and the stress due to local bending; but in no case should the grip length of the rivet exceed eight times its diameter.

(2) Holes for rivets shall be fair and true, and, as far as practicable, at right angles to the face of the metal, and both heads of the rivet shall be concentric with the shank.

(3) Rivets shall be snapped with a cup head, except when countersunk. Cupheads shall not be less in height than three-quarters of the diameter of the shank, and not less in diameter than one and a-half times the diameter of the shank, plus one-eighth of an inch.

(4) Countersunk rivets shall be used only where really necessary, and in such cases a taper of 60 degrees shall be worked to in the countersinking; while the depth of such countersinking shall be at least equal to half the diameter of the rivet. Countersunk rivets must after driving project above the surface. In cases where the work must be flush, the projections of the countersunk rivets can be chipped off after riveting. Ordinarily they should be left projecting one-eighth of an inch, and only be cut when absolutely necessary.

(5) Rivet holes shall be either drilled (after clamping together the different thicknesses) from the solid to the full size of the finished hole; or the individual thicknesses may be punched separately to a size one-quarter of an inch less than the size of the finished hole. The diameter of the finished hole shall not exceed that of the rivet shank by more than one-sixteenth of an inch. The diameter of the hole shall be the diameter of the rivet calculated. Punched holes shall be drilled out true to the full size after the different thicknesses have been clamped together. Any burrs, including those between the different thicknesses, must be taken off. When approved by the District Surveyor holes in minor details may be punched full size. The whole length of the rivet shall be heated to " cherry red " before driving, care being taken not to burn the rivet.

(6)The spacing of rivets in compression members shall not exceed six inches.

(7) In order that rivets in plate and compound girders may not be stressed beyond the limits permitted by Section 22 (22) (23) the spacing shall be determined in the following manner:—

$$S \cdot p = n \cdot R \cdot d \text{ or } p = n \cdot R \cdot d/S$$

Where:

S = Total vertical shear at the section under investigation.

R = *Resistance* to shear of one rivet.

p = *Pitch* or spacing of rivets.

n = *Number* of longitudinal rows of rivets.

d = *Depth*, measured as follows: For the web—distance between centres of rivets. For the flanges—distance measured over angles or (in the case of compound girders) over the joist.

PROTECTING CASING OF PILLARS.—Section 22 (3) (5).

Where a pillar is for part of its height situated in an external wall of a building, and for part of its height situated within the building, such part of the height as is situated in the external wall shall be protected in the manner provided in subsection (3), and such part as is situated within the building in the manner provided in subsection (5).

TWIN BEAMS.—Section 22 (18).

Where twin beams are used, and can be spaced apart so as to enable concrete to be filled between, gas barrels and bolts are sufficient as separators; but where the beams are close together cast iron separators should be employed.

BOLTING.—Section 22 (10).

Where riveting is impracticable, and bolts have to be used, the unthreaded shank of every bolt must extend through the parts connected, and be provided with a washer to enable the nut to be thoroughly tightened.

SOLID STEEL PILLARS.—Section 22 (12).

Section 22 (12) precludes the use of solid steel pillars throughout a building, but they may be used in isolated situations, subject to the following conditions:—

(a) Solid steel pillars shall not be considered as adding to the stability of the building, and sufficient rigidity of the building shall be provided by other means.

(b) The thickness of the cap shall not be less than one half the diameter of the pillar; but, unless required by paragraph (j), (k) and (l) need not exceed 4 inches.

(c) The minimum width of the slab forming the cap or base shall not be less than twice the diameter of the pillar.

(d) In the case of super-imposed pillars, the girder shall rest upon the base of the upper pillar, and not upon the cap of the lower pillar; and the seating of the girder shall be close up to the shaft of the pillar.

(e) The pillar shall not be subjected to a higher stress than that set out in the table for pillars with one end fixed and one end hinged. (See Sec. 22 (21).)

(f) The solid pillars shall be calculated like other pillars as regards length, i.e., from floor level to floor level.

(g) In a pillar extending through two storeys, if the pillar length be taken from floor to floor, the connections shall be capable of developing a resistance moment equal to the moment induced by the eccentric loading at that connection. In case the connection should not be strong enough to do this, the pillar length shall be measured as the full height from cap to base through both storeys.

(h) For an intermediate connection in a two-storey length, the collar straps shall be riveted to the web of the girders by not less than four three-quarter inch diameter rivets to each. The collar straps shall not be less than half an inch thick.

(i) Solid steel pillars should be true and flush at their ends, and be machined at right angles to the axis of the pillar after the caps and bases have been shrunk on.

(j) The shoulder at each end shall have sufficient area to carry the load on the pillar at a stress not exceeding 11 tons per square inch.

(k) The fibre and bearing stresses induced in the caps and bases by flexure (with each considered as independent of the other) shall not exceed 5 and 11 tons per square inch respectively.

(l) The fibre and bearing stresses induced in the neck by flexure shall not exceed 5 and 11 tons per square inch respectively.

(m) In calculating the moment of flexure the lever arm shall be taken as the distance from the centre of the bearing surface supporting the load applied to the centre of the pillar.

(n) The extra stress due to any eccentric load shall be taken up in the section below the beam.

(o) The caps and bases shall be connected with at least four bolts not less than three-quarters of an inch in diameter. The group of connecting bolts shall have a minimum section modulus equal to that of the shaft of the pillar.

(p) In the case of intermediate connections, the webs of the beams shall always be tightly connected to the pillar by straps bolted to the beam.

JOINTS IN AND BETWEEN PILLARS.—Sec. 22 (12) b.

(See Appendix IV.)

The expression, " joints in such pillars " shall be considered to refer to joints in the length of a pillar that passes through several storeys, where the same or similar sections are employed above and below the joint in question, with or without additional plates riveted to the lower length.

The expression " joints between such pillars " shall be considered to refer to the case where the pillar above the joint in question is of a substantially different form or section from that below the joint; as in the case where a single I section is super-imposed on a double I section.

For joints in pillars, the dimensions of the splice plates shall be determined in the following manner:—

Let:

t_s = thickness of splice plate.

b = breadth or width of splice plate.

d_s = distance, centre to centre, of splice plates.

W = load on bracket supporting the beam next above the joint.

e = its arm of eccentricity (not less than $\frac{d}{2}$)

d = depth or width of pillar over flanges.

Then, in the case of butted and spliced joints, for splice plates for single and compound sections:

When the load is applied on the flange side of the pillar b must be equal to the width of the pillar flange.

$$t_s = \frac{We}{7 \cdot 5\ b\ d_s}$$

When the load is applied on the web side of the pillar

$$t_s = \frac{We}{2 \cdot 5\ b^2}$$

And, in the case of fully spliced joints, the splice plates shall be deemed to form part of the pillar, and the combined stresses due to bending and direct loading shall not exceed the stress allowed by Sec. 22 (21) a, as calculated by the formula:—

$$p = \frac{P}{A} + \frac{B}{M}$$

where:

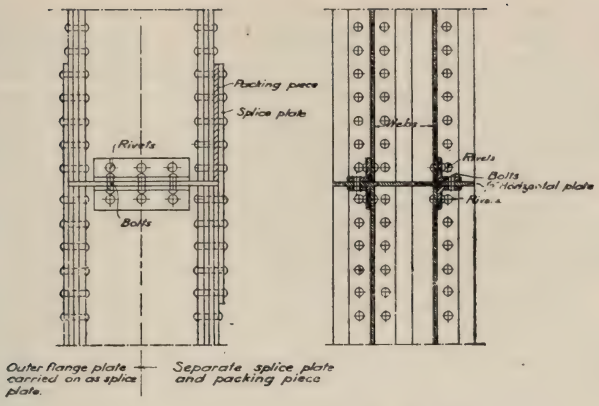
p = pillar stress or intensity of pressure.
B = Bending moment calculated as aforesaid.
M = Section modulus of splice plates only, about the axis under consideration.
P = Total direct load or pressure.
A = Combined area of both splice plates.

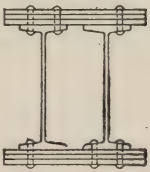
Web splice plates for single web sections shall be of sufficient area to resist a shear equal to the accumulated wind pressure at the joint under consideration, and no splice plate shall be of less thickness than three-eighths of an inch.

For double web sections the connections shall be made as shown on the sketch adjoining, with angles not less than $3\frac{1}{2}$ in. x $3\frac{1}{2}$ in. x $\frac{1}{2}$ in., and of the full width of the web, with sufficient bolts or rivets to withstand the shear above referred to.

When beams are connected to a pillar on its opposite faces the algebraic sum of the moments of the eccentric loads minus the moment of the superimposed load on the more lightly loaded beam shall be taken in ascertaining the bending moment.

When beams are connected to two adjoining, or to three or to four faces of the pillar, the algebraic sum of the moments in each direction shall be calculated in the foregoing manner; and the splice plates so dimensioned that the combined stresses at any point in their breadth shall not exceed $7\frac{1}{2}$ tons per square inch.





Connection for Double Web Sections.

A sufficient number of rivets shall be used in each splice plate on each side of the joint to withstand the stresses calculated in the foregoing manner, but in no case shall the number on each side of the joint be less than eight.

In order that a joint between pillars shall be considered to be "properly fixed and made," it shall comply with either of the following sets of conditions:—

(1) Whenever practicable the joint shall have splice plates to each flange, together with any necessary butting plates or filling plates.

(a) In the case of a butted and spliced joint, the bearing surfaces of the upper and lower ends of the pillars, including any necessary butting plates on the upper pillar (which shall be connected to such pillar with a sufficient number of rivets to carry the loads transmitted by such plates) shall, after assembly and riveting, be machined true and normal to the axis of the pillar, so as to secure an even and true bearing; and the area of the surfaces in bearing shall be not less than the sectional area of the pillar above.

(b) In the case of a fully spliced joint, the load shall be transmitted from the upper to the lower pillar (with any necessary filling plates), wholly through splice plates of

sufficient sectional area, and with a sufficient number of rivets, to resist safely the whole of the loads on the pillar.

In either case, the splice plates shall be of not less thickness than half an inch, and be of sufficient sectional area to resist safely the shearing stresses due to the loads and wind pressure; and shall be constructed and fixed in the manner set out for splice plates to joints in pillars.

(2) Where the upper and lower pillars are of sections so dissimilar that the above method is not practicable, the following method shall be adopted:—

The lower end of the upper pillar and the upper end of the lower pillar shall each (after assembly and riveting) be machined true and normal to the axis of the pillar. A horizontal plate or plates of dimensions on plan at least equal to the over-all dimensions ot the top of the lower pillar and its cleats, and of sufficient thickness to safely carry the load (in no case less than three-quarters of an inch) shall be inserted between the pillars, and riveted in the first instance to the foot of the upper pillar, with sufficient angle cleats and gusset pieces, and sufficient rivets, to properly distribute the load on to the horizontal plate, due allowance being made for the bearing of any machined surfaces. The upper pillar and the horizontal plate or plates shall then, with a sufficient number of rivets, be riveted to the upper end of the lower pillar, provided with sufficient angle cleats riveted thereto with a sufficient number of rivets, so that, as far as possible, the cleats of the upper pillar shall be riveted (through the horizontal plate or plates) to the cleats of the lower pillar.

BASE PLATES.—Section 22 (12) c.

Base plates may be constructed in one of the following ways, inter alia:—

(1) A solid steel slab of sufficient thickness and area to comply with the requirements of the Act as to stresses. The load may be assumed to be carried directly from the pillar shaft into the steel slab. In such cases the pillar shaft shall be provided with angles with the bearing surfaces machined and bolted down to the steel slab with at least four bolts.

(2) A properly designed cast iron or cast steel base machined true and bolted down as required in the previous paragraph.

(3) In the case of a pillar of which the lower end has been machined true after riveting, and when part of the load is transmitted by bearing stress directly to the metal below, a sufficient number of rivets shall be provided to the gusset pieces to transmit the remainder of the load by shearing stress in the horizontal rivets. It shall be assumed that the bearing pressure on the base of the pillar is evenly distributed, and the base plates and gussets shall be calculated accordingly, but in no case shall the base plate be less than three-quarters of an inch in thickness.

INTERPOSED METAL.—Section 22 (15).

As regards Section 22 (15) dealing with the stress in metal interposed between the ends of a superimposed pillar and the pillar beneath, in the case of an interposed beam, the length of the web to be taken into account shall be equal to the width of the shaft of the pillar above, plus (a) in the case of a continuous beam, the depth of such beam; and (b) in the case of a non-continuous beam, one-half of the depth of such beam. Provided that the slenderness ratio of the web together with any stiffeners at that point do not exceed the slenderness ratio of the upper pillar.

LOADS ON WAREHOUSE FLOORS.—Section 22 (18) c.

Section 22 (18) c shall be interpreted to mean that in a warehouse building floors may be designed for lighter loads than 2 cwt. per square foot, if used for purposes other than warehousing.

BRICK PIERS.—Section 22 (18) c.

The brick piers referred to in this subsection may be in the foundations up to the lowest floor level, or in party walls.

WIND BRACING.—Section 22 (20).

Diagonal bracing in the vertical planes need not be inserted in cases where the walls and floors give sufficient rigidity to resist the deformation of the building due to wind pressure.

In all cases where the walls, floors and other internal construction do not render the building sufficiently rigid to resist safely the wind pressure specified in Section 22 (20),

then, cleated joints, knee bracing or diagonal bracing shall be provided to the steel frame in order to comply with the subsection.

FIXITY OF ENDS OF PILLARS.—Section 22 (21).

Absolute fixity of ends of pillars is seldom, if ever, attained in actual practice; but pillars that comply with the following conditions may, inter alia, for practical purposes, be considered as fixed:—

(1) The pillar shall be continuous throughout its whole height, and, if not rolled in one length, shall be jointed at or near the level of the beam attached to the pillar, either with (a) a closely butted and spliced joint, or (b) a fully spliced joint.

(2) The joint shall in all cases have splice plates to each flange of not less thickness than half an inch, and of sufficient sectional area to resist safely the bending moment and shearing stresses due to the loads and wind pressure. Single web sections shall have in addition two splice plates to the web. The rivets in the splice plates shall be not less than three-quarters of an inch in diameter, and not less in number in each flange than eight above and eight below the joint. Where the upper and lower lengths are not of the same sectional form, and in the case of hollow pillars, in addition to the splice plates on the flanges, a horizontal plate of sufficient thickness (in no case less than half an inch) shall be inserted between the upper and lower lengths, and be connected therewith by rivets or bolts.

(a) In the case of a butted and spliced joint, the load shall be transmitted from the upper to the lower length wholly by the bearing stress of the surfaces in contact. The ends of the lengths, including any butting plates (which shall be connected by a sufficient number of rivets to carry the load transmitted to such plates), shall, after assembly and riveting, be machined true and normal to the axis of the pillar, so that the upper length shall bear evenly and closely on the lower.

(b) In the case of a fully spliced joint, the load shall be transmitted from the upper to the lower length wholly through splice plates of sufficient sectional area and a sufficient number of rivets to resist the whole of the loads on the pillar, and the bending moment induced thereby.

(3) The pillar shall have attached thereto in the direction, on plan, of the radius of gyration under consideration,

and in the manner set out, a beam connected to the pillar by closely fitted angle cleats above and below, in addition to any necessary gusset brackets. Such cleats shall be riveted to both pillar and beam by rivets of not less diameter than three-quarters of an inch. In any one leg of such cleats there shall not be fewer than two rivets, and the total sectional area of the rivets in any one leg shall not be less than the area computed by the following formula:—

$$A_r = \frac{1}{4} \cdot A_f \cdot \frac{d_p}{d_b}$$

A_r = Total sectional area of rivets

A_f = Sectional area of one flange of the beam

d_p = Diameter or thickness of pillar in the direction of beam

d_b = Depth of beam

Such cleat shall be of not less thickness than one quarter of an inch less than the diameter of the rivets used, but in no case less than half an inch thick.

In the case of a flange connection the cleats shall extend to the full width of the beam; and, in the case of a web connection to the full width of the flat portion of the web.

Any additional rivets required to resist the shear due to the loading of the beam must also be provided in the bottom cleat.

Pillars that comply with the foregoing conditions may be considered as having both ends fixed for each section of their length comprised within the limits of the vertical distance between the upper surfaces of two girders so attached.

When two beams are attached to the pillar at or about the same level in the direction of the radius of gyration under consideration the method of fixing in that direction shall be determined by the beam of greater efficiency.

The condition of fixity of a pillar in the direction of each attached beam may be considered independently; but the load to which such pillar may be subjected shall be regulated by the minimum result of the various calculations for conditions of fixing in such direction.

A section shall not be permitted under the highest stress in a butt joint unless it has complete bearing on the section below.

A pillar with either twin beams passing one on each side of it, or a pillar consisting of a twin section with a beam passing through it, shall be considered as having a fixed end, provided that the junction shall be at least of equal strength to that previously described.

ECCENTRIC LOADING OF PILLARS.—Section 22 (21)e.

For making calculations as to the effect of eccentric loading of pillars, when the load is placed on one or other of the rectangular axes, the following general formulæ are advocated:—

For bending moment:

$$B = P' e \qquad (1)$$

But for a continuous pillar in all but the top length:

$$B = \frac{P' e}{2} \qquad (1a)$$

For extreme fibre stress:

$$f = \frac{B n}{I} = \frac{B}{K} \qquad (2)$$

For maximum stress induced:

$$p = \frac{P}{A} + f \qquad (3)$$

For sectional area of pillar:

$$A = \frac{P}{p} + \frac{B n}{p g^2} \qquad (4)$$

Where:

B = Bending moment, in inch tons, due to eccentric load.
e = Arm of eccentricity, in inches.
n = Distance of extreme fibre stress from neutral axis, in inches.
f = Extreme fibre stress due to eccentric load, in inch tons per square inch of section.
A = Total area of cross section, in square inches.
M = Section modulus, in inches^3.
P = Total load on pillar, both axial and eccentric.

P′ = Pressure due to eccentric load.

p = Maximum stress, in tons per square inch of section.

g = Radius of gyration of section, in inches.

The value of *e* shall be taken as follows:—

For single-web connections:—

One-half the projection of the cleat or 2 inches, whichever is the greater.

For double-web connections, and

For flange connections:—

One-half the thickness or diameter of the pillar measured in the direction of the length of the beam.

As the buildings should be sufficiently braced to ensure that no material deflection occurs in the pillars, the above formula (1) has been adopted in preference to the more complicated one necessary where material deflection occurs.

When an eccentric load is not placed on the rectangular axis, the total flexural stress shall be calculated as the algebraic sum of the flexural stresses obtained by considering in turn the arm of eccentricity about each of the rectangular axes. The eccentricity for that purpose shall be taken as the distance of the point of application from each rectangular axis of the pillar; but the eccentric arm in either direction shall in no case be taken at less than 2 inches.

In cases where the cleat is not supported by a vertical stiffener, the eccentricity of the web connection may be taken at one inch; but in all such cases the pillar shall be deemed to be hinged at this connection, and efficient wind bracing shall be provided.

As an alternative to the foregoing method of calculating the bending moment due to eccentricity in continuous pillars, the full value of the eccentric arm may be taken, but in that case the sections of the pillar above and below the connection may be taken as conjointly resisting the bending moment.

WEIGHTS OF MATERIALS.

When special tests are not made in respect of the weights of the particular materials to be used on a given building, the weights to be allowed for shall be in accordance with the following table of average weights:—

AVERAGE WEIGHTS OF MATERIALS GENERALLY.

Material.	Average weight in pounds per cubic foot.	Average weight in pounds for a slab one foot square and one inch thick.
Wood.		
Northern pine (including Baltic pine or fir)	36	3
Pitch pine (including American long leaf yellow pine) ...	48	4
Oak, teak, maple and other constructional hard woods ...	54	4·5
Brickwork, Masonry, &c.		
Asphalt	144	12
Breeze Concrete	90	7·5
Brick Concrete...	112	9·3
Breeze and sand Concrete ...	112	9·3
Brick and sand Concrete ...	120	10
Flint or Thames ballast Concrete	140	11·7
Blue or Glazed or similar brickwork in lime or cement mortar	168	14
Brickwork in lime or cement mortar (stocks, Flettons or similar)	112	9·3
Brickwork faced with stone, or stone backed with brickwork	140	11·7
Plastering on walls and ceilings	96	8
Cement floating on floors, &c....	144	12
Granitic finish...	168	14
Stone, marble or granite ...	168	14

Metals.

Metal.	Specific Gravity.	Average weight in pounds per cubic foot.	Average weight in pounds for a slab one foot sq., 1 inch thick.	Average weight of a bar one inch square and one foot long.
Cast Iron (average weight for ordinary calculations)	7·22	461	38·4	3·2
Wrought Iron do. ...	7·7	480	40	3·3
Cast Steel do. ...	7·86	490	40·8	3·4
British Standard Mild Steel	7·86	490	40·8	3·4

APPENDIX I.

STRESSES IN MANSARD ROOFS.

By H. Kempton Dyson.

In the construction of Mansard roofs cranked girders are frequently employed, and belong generally to the types

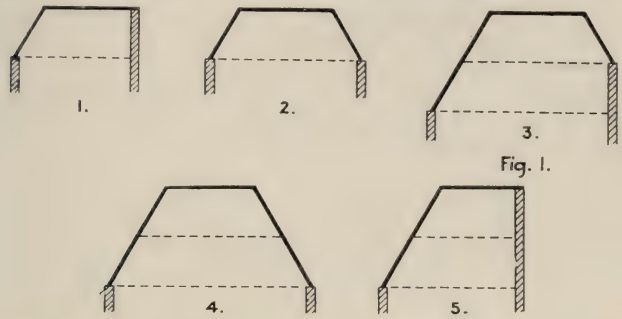


Fig. 1.

shown in Fig. 1. Types 1 and 2, applying to roofs of one storey, are common. Types 3, 4 and 5, applying to two storeys, might be extended by similar forms to roofs of three or more storeys. In London the Building Acts limit the number of storeys in a roof to two, but it is rather exceptional to have cranked girders extending through two storeys. Some of the first and fifth types have been erected from the designs of the writer in an office building in Dartmouth Street, Westminster, the intermediate floor girders in those of Type 5 resting on the cranked girders.

It will be obvious that, if cranked girders merely rest upon supports (such as brick walls) which are incapable of supplying any lateral abutment, the reactions at their ends will be vertical, and in consequence there will be a bending moment at the knuckle. If a cranked girder of Type 2 rests upon and is bolted to a girder at its feet a lateral abutment is provided and a horizontal thrust induced, so that the cranked girder becomes somewhat like an arch. Owing to the exigencies of practical construction it is not usual to afford complete fixity at the feet of such girder, though in steel-frame construction it would be quite possible to do so, even with girders of Types 1 and 5, as sketched in Fig. 2,

in which case there would be considerable bending moments at points (a), (b), (c) and (d).

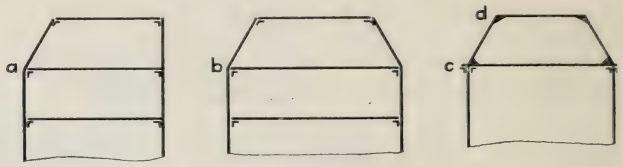


Fig. 2

When the horizontal and inclined members are of one and the same piece the resistance to bending is fairly simply obtained; but, as they frequently are separate, the joint at the knuckle requires special treatment. Thus, if a joint were made in two steel joist sections, such as sketched in Fig. 3 (a), and subjected to a bending moment, B it would tend to be distorted as Fig 3 (b).

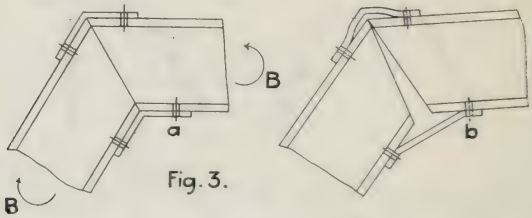


Fig. 3.

There are several ways in which such a joint can be properly made.

The forces to which roofs in which such cranked beams occur are subjected are the dead weight of the construction and superimposed live loads from goods and persons, wind pressure and snow. A difference in angle of the roof (which, under the London Building Act of 1894, must not exceed 47 degrees in the case of buildings of the warehouse class and 75 degrees in other buildings) will oppose a different resistance to the stream line flow of the wind, and unfortunately there is insufficient experimental data at present to afford us proper information as to the intensity of pressure upon the surfaces of Mansard roofs. All we know is that the pressure is not uniform, and is positive to windward and negative (i.e., suction) to leeward.

The only mathematical treatment of roofs of the Mansard type that the writer has seen is in the Third Edition of Marsh and Dunn's Manual of Reinforced Concrete, and derived from a German source. It is not sufficiently general, and is not adequate in that it appears to be derived by

equating slopes and deflections at the joints, without regard to the effect of thrust and shear in deforming the structure.

Some simple cases will suffice to show the more rigid treatment by Castigliano's "Theorem of Least Work," the following assumptions being made:—

(1) That the cranked girder is not fixed at the feet but merely hinged. (The exigencies of practice make this assumption generally warrantable.)

(2) That the feet are maintained rigidly in their original position by being tied or propped so as to prevent the feet escaping laterally. (The bolting—preferably riveting—of the feet should be carefully attended to, in order to see that this condition is properly secured. The extension of the girder tying the feet due to the tension induced is so small as to be safely neglected, and it is offset by the deflection of such girder by its own load.)

For the purpose of designing it is suggested that, where the loading approximates to that dealt with, the horizontal force should be roughly determined by the short approximate formulæ given (in the case of several loads the horizontal force will be the sum of the forces induced by each load considered separately), and, after the section has been chosen to suit, checked by the longer formulæ set out.

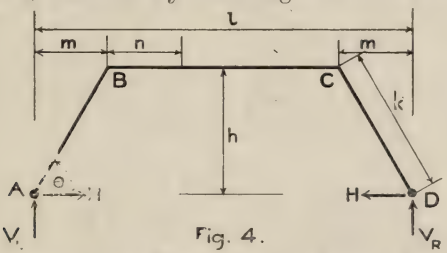


Fig. 4.

NOMENCLATURE AND NOTATION (Referring to Fig. 4).

	Total sectional area.	Joist web area.	Moment of Inertia.	Thrust.
$\overline{A\,B}$ = Left leg, having	A_1	a_1	I_1	T_1
$\overline{C\,D}$ = Right leg, having	A_2	a_2	I_2	T_2
$\overline{B\,C}$ = Top, having	A_3	a_3	I_3	T_3

h = height

l = length of span

k, m and n = other lengths

x = any distance measured from either end

E and E_s = moduli of elasticity for tension or compression and shearing

P = total pressure or load

V_L and V_R = vertical reactions at left and right

H = horizontal force at abutment

T = thrust, generally
S = shearing force
B = Bending Moment
T_x S_x and B_x = thrust, shearing force and Bending Moment at x
W = total work resulting from deformation of structure
W_t W_s W_b = work due to thrust, shear and bending

CASE 1.

Concentrated load at any point on top of girder (Fig. 5).

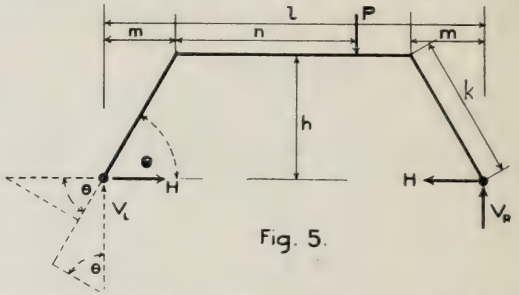


Fig. 5.

Left leg (measuring x to the right up the incline):

$$V_L = P\left(1 - \frac{m}{l} - \frac{n}{l}\right)$$

$$T_1 = H\cos\theta + V_L \sin\theta = \frac{Hm + V_L h}{k}$$

$$W_T = \int \frac{T_1^2}{2A_1E} \cdot dx$$

$$\frac{dW_T}{dH} = \frac{d}{dH}\int \frac{T_1^2}{2A_1E}\cdot dx = \int \frac{2T_1}{2A_1E}\cdot\frac{dT_1}{dH}\cdot dx$$

$$= \int \frac{T_1}{A_1E}\cdot\frac{dT_1}{dH}\cdot dx = \int_0^k \frac{T_1}{A_1E}\cdot\frac{m}{k}\cdot dx$$

$$= \int_0^k \frac{Hm^2 + V_L hm}{A_1Ek^2}\cdot dx = \frac{Hm^2 V_L hm}{A_1Ek}$$

$$S_x = V_L\cos\theta - H\sin\theta = \frac{V_L m - Hh}{k}$$

$$W_S = \int \frac{S_x^2}{2\alpha_1 E_s}\cdot dx$$

$$\frac{dW_S}{dH} = \int \frac{S_x}{\alpha_1 E_s}\cdot\frac{dS_x}{dH}\cdot dx = \int_0^k -\frac{V_L hm - Hh^2}{\alpha_1 E_s k^2}\cdot dx = \frac{Hh^2 - V_L hm}{\alpha_1 E_s k}$$

$$B_x = (V_L\cos\theta - H\sin\theta)x = \frac{(V_L m - Hh)x}{k}$$

$$W_B = \int \frac{B_x^2}{2EI_1}\cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_1}\cdot\frac{dB_x}{dH}\cdot dx = \int_0^k -\frac{h}{EI_1 k}\left[\frac{(V_L m - Hh)}{k}x^2\right]dx$$

$$= -\frac{hk}{3EI_1}(V_L m - Hh)$$

Right leg (measuring x to the left up the incline):

$$V_R = P\,\frac{(m+n)}{l}$$

$$T_2 = -H\cos\theta + V_R \sin\theta = -\frac{Hm + V_R h}{k}$$

$$W_T = \int \frac{T_2^2}{2A_2E}\cdot dx$$

$$\frac{dW_T}{dH} = \int \frac{T_2}{A_2E}\cdot\frac{dT_2}{dH}\cdot dx = \int_0^k \frac{T_2}{A_2E}\cdot\frac{m}{k}\cdot dx$$

$$= -\int_0^k \frac{Hm^2 + V_R hm}{A_2 E k^2}\cdot dx = -\frac{Hm^2 + V_R hm}{A_2 E k}$$

$$S_x = V_R\cos\theta - H\sin\theta = -\frac{V_R m - Hh}{k}$$

$$W_S = \int \frac{S_x^2}{2\alpha_2 E_s}\cdot dx$$

$$\frac{dW_S}{dH} = \int \frac{S_x}{\alpha_2 E_s}\cdot\frac{dS_x}{dH}\cdot dx = \int_0^k \frac{S_x}{\alpha_2 E_s}\cdot\left(-\frac{h}{k}\right)\cdot dx$$

$$= \int_0^k \frac{Hh^2 - V_R hm}{\alpha_2 E_s k^2}\cdot dx = \frac{Hh^2 - V_R hm}{\alpha_2 E_s k}$$

$$B_x = (V_R\cos\theta - H\sin\theta)x = -\frac{(V_R m - Hh)}{k}\cdot x$$

$$W_B = \int \frac{B_x}{2EI_2}\cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_2}\cdot\frac{dB_x}{dH}\cdot dx = \int_0^k -\frac{h}{EI_2 k}\left[\frac{(V_R m - Hh)}{k}x^2\right]dx$$

$$= -\frac{hk}{3EI_2}(V_R m - Hh)$$

Top left portion (measuring x horizontally to the right):

$$T_3 = H$$

$$W_T = \int_0^n \frac{T_3^2}{2A_3E}\cdot dx = \frac{T_3^2 n}{2A_3E} = \frac{H^2 n}{2A_3E}$$

$$\frac{dW_T}{dH} = \frac{Hn}{A_3E}$$

$$S_x = V_L$$

$$W_S = \int_0^n \frac{S_x^2}{2\alpha_3 E_s}\cdot dx = \frac{S_x^2 n}{2\alpha_3 E_s}$$

$$\frac{dW_S}{dH} = 0$$

$$B_x = V_L x - Hh$$

$$W_B = \int \frac{B_x^2}{2EI_3} \cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_3} \cdot \frac{dB_x}{dH} \cdot dx = \int_m^{m+n} -\frac{h}{EI_3}(V_L x - Hh)\,dx$$

$$= \frac{h}{EI_3}\left[V_L \frac{m^2}{2} - Hhm - \frac{V_L (m+n)^2}{2} + Hh(m+n) \right]$$

$$= \frac{h}{EI_3}\left[-\frac{V_L}{2}(2mn + n^2) + Hhn \right]$$

Top right portion (measuring x horizontally to the left):

$$T_3 = H$$

$$W_T = \int_0^{l-2m-n} \frac{T_3^2}{2A_3E} \cdot dx = \frac{T_3^2}{2A_3E}(l-2m-n) = \frac{H^2}{2A_3E}(l-2m-n)$$

$$\frac{dW_T}{dH} = \frac{H}{A_3E}(l-2m-n)$$

$$S_x = V_R$$

$$\frac{dW_S}{dH} = 0$$

$$B_x = V_R x - Hh$$

$$W_B = \int \frac{B_x^2}{2EI_3} \cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_3} \cdot \frac{dB_x}{dH} \cdot dx = \int_m^{l-m-n} -\frac{h}{EI_3}(V_R x - Hh)\,dx$$

$$= \frac{h}{EI_3}\left[V_R \frac{m^2}{2} - Hhm - V_R \frac{(l-m-n)^2}{2} + Hh(l-m-n) \right]$$

$$= \frac{h}{EI_3}\left[-\frac{V_R}{2}(l^2 - 2lm - 2ln + 2mn + n^2) + Hh(l-2m-n) \right]$$

Totalling $W = \Sigma W_T + \Sigma W_S + \Sigma W_B$

and $\frac{dW}{dH} = \Sigma \frac{dW_T}{dH} + \Sigma \frac{dW_S}{dH} + \Sigma \frac{dW_B}{dH}$

Therefore putting $d\,W/d\,H=O$ in order to determine the "least work" we have

$$H\left[\frac{m^2}{Ek}\left(\frac{1}{A_1}+\frac{1}{A_2}\right)+\frac{(l-2m)}{A_3E}+\frac{h^2}{E_3k}\left(\frac{1}{\alpha_1}+\frac{1}{\alpha_2}\right)+\frac{h^2k}{3E}\left(\frac{1}{I_1}+\frac{1}{I_2}\right)+\frac{h^2}{EI_3}(l-2m)\right]$$
$$=-P\left[\frac{hm}{Ek}\left\{\frac{1}{A_1}\left(1-\frac{m}{l}-\frac{n}{l}\right)+\frac{1}{A_2}\left(\frac{m+n}{l}\right)\right\}-\frac{hm}{E_3k}\left\{\frac{1}{\alpha_1}\left(1-\frac{m}{l}-\frac{n}{l}\right)+\frac{1}{\alpha_2}\left(\frac{m+n}{l}\right)\right\}\right.$$
$$\left.-\frac{hkm}{3E}\left\{\frac{1}{I_1}\left(1-\frac{m}{l}-\frac{n}{l}\right)+\frac{1}{I_2}\left(\frac{m+n}{l}\right)\right\}-\frac{h}{2EI_3}\left\{(m+n)(l-2m)-n^2\right\}\right]$$

For an approximate formula we can disregard the effect of the thrust and shear, and we may also assume the section to be uniform throughout, so that E and I will cancel out. Thus:—

$$Hh^2\left[\frac{2}{3}k+(l-2m)\right]=Ph\left[\frac{km}{3}+\frac{(m+n)}{2}(l-2m)-\frac{n^2}{2}\right]$$

$$\text{or}\quad H=\frac{P}{2h}\left[\frac{2km+3(m+n)(l-2m)-3n^2}{2k+3(l-2m)}\right]$$

Then for a central load, where $m+n=\frac{l}{2}$

$$H=\frac{P}{8h}\left[\frac{8km+3(l-2m)(l+2m)}{2k+3(l-2m)}\right]$$

For a load applied at the crank, where $n=O$

$$H=\frac{Pm}{2h}$$

CASE 2.—Distributed load on top of girder (Fig. 6).

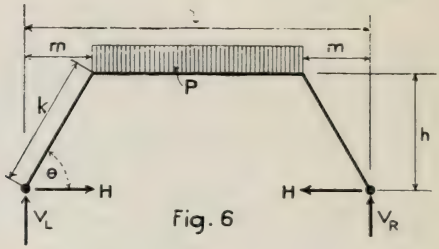


Fig. 6

Left leg (measuring x to the right up the incline):

$$V_L = V_R = \frac{P}{2}$$

$$T_1 = H\cos\theta + V_L\sin\theta = \frac{Hm + V_L h}{k}$$

$$W_T = \int \frac{T_1^2}{2A_1E}\cdot dx$$

$$\frac{dW_T}{dH} = \int \frac{T_1}{A_1E}\cdot\frac{dT_1}{dH}\cdot dx = \int_0^k \frac{T_1}{A_1E}\cdot\frac{m}{k}\cdot dx$$

$$= \int_0^k \frac{Hm^2 + V_L hm}{A_1Ek^2}\cdot dx = \frac{Hm^2 + V_L hm}{A_1Ek}$$

$$S_x = V_L\cos\theta - H\sin\theta = \frac{V_L m - Hh}{k}$$

$$W_S = \int \frac{S_x^2}{2a_1E_s}\cdot dx$$

$$\frac{dW_S}{dH} = \int \frac{S_x}{a_1E_s}\cdot\frac{dS_x}{dH}\cdot dx = \int_0^k -\frac{V_L hm - Hh^2}{a_1E_sk^2}\cdot dx = \frac{Hh^2 - V_L hm}{a_1E_sk}$$

$$B_x = (V_L\cos\theta - H\sin\theta)x = \frac{V_L m - Hh}{k}\cdot x$$

$$W_B = \int \frac{B_x^2}{2EI_1}\cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_1}\cdot\frac{dB_x}{dH}\cdot dx = \int_0^k -\frac{h}{EI_1k}\left(\frac{V_L m - Hh}{k}\right)x^2 dx$$

$$= -\frac{hk}{3EI_1}(V_L m - Hh)$$

Right leg (measuring x to the left up the incline):
Similarly to left leg.

$$\frac{d W_T}{d H} = \frac{H m^2 + V_R h m}{A_2 E k}$$

$$\frac{d W_S}{d H} = \frac{H h^2 - V_R h m}{\alpha_2 E_s k}$$

$$\frac{d W_B}{d H} = \frac{h k}{3 E I_2} (V_R m - H h)$$

Top (measuring x horizontally to the right):

$$T_3 = H$$

$$W_T = \int_0^{l-2m} \frac{T_3^2}{2 A_3 E} \cdot dx = \frac{T_3^2}{2 A_3 E} (l-2m) = \frac{H^2}{2 A_3 E} (l-2m)$$

$$\frac{d W_T}{d H} = \frac{H}{A_3 E} (l-2m)$$

$$S_x = V_L - \frac{P(x-m)}{(l-2m)} = \frac{P}{2} \frac{(l-2x)}{(l-2m)}$$

$$W_S = \int \frac{S_x^2}{2 \alpha_3 E_s} \cdot dx$$

$$\frac{d W_S}{d H} = 0$$

$$B_x = V_L x - H h - \frac{P(x-m)^2}{2(l-2m)} = \frac{P}{2} x - \frac{P}{2} \frac{(x-m)^2}{(l-2m)} - H h$$

$$W_B = \int \frac{B_x^2}{2 E I_3} \cdot dx$$

$$\frac{d W_B}{d H} = \int \frac{B_x}{E I_3} \cdot \frac{d B_x}{d H} \cdot dx = \int_m^{l-m} \frac{h}{E I_3} \left[\frac{P}{2} x - \frac{P(x-m)^2}{2(l-2m)} - H h \right] dx$$

$$= \frac{h}{E I_3} \left[\frac{P}{2} \left\{ \frac{m^2}{2} - \frac{\left(\frac{m^3}{3} - \frac{2m^3}{2} + m^3\right)}{(l-2m)} \right\} - H h m \right.$$

$$\left. - \frac{P}{2} \left\{ \frac{(l-m)^2}{2} - \frac{\frac{(l-m)^3}{3} - \frac{2m}{2}(l-m)^2 + m^2(l-m)}{(l-2m)} \right\} + H h (l-m) \right]$$

$$= \frac{h}{E I_3} (l-2m) \left[-\frac{P}{12} (l+4m) + H h \right]$$

Totalling and putting $d\,W/d\,H = 0$ we have

$$H\left[\frac{m^2}{Ek}\left(\frac{1}{A_1}+\frac{1}{A_2}\right)+\frac{(l-2m)}{A_3E}+\frac{h^2}{E_3k}\left(\frac{1}{\alpha_1}+\frac{1}{\alpha_2}\right)+\frac{h^2k}{3E}\left(\frac{1}{I_1}+\frac{1}{I_2}\right)+\frac{h^2(l-2m)}{EI_3}\right]$$
$$=P\left[-\frac{hm}{2Ek}\left(\frac{1}{A_1}+\frac{1}{A_2}\right)+\frac{hm}{2E_3k}\left(\frac{1}{\alpha_1}+\frac{1}{\alpha_2}\right)+\frac{hkm}{6E}\left(\frac{1}{I_1}+\frac{1}{I_2}\right)+\frac{h(l-2m)}{12EI_3}(l+4m)\right]$$

For an approximate formula we can disregard the effect of thrust and shear, and also assume the section to be uniform throughout.

Thus:—

$$H=\frac{P}{12h}\left[\frac{4mk+(l-2m)(l+4m)}{\frac{2}{3}k+(l-2m)}\right]$$

CASE 3.—Distributed load on one leg (Fig. 7).

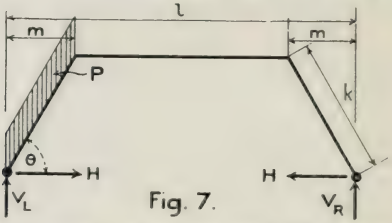


Fig. 7.

Left leg (measuring x to the right up the incline):

$$V_L=\frac{P(l-\frac{m}{2})}{l}=P\left(1-\frac{m}{2l}\right)$$

$$T_x=H\cos\theta-V_L\sin\theta-\frac{P}{k}x\sin\theta=\frac{Hm+V_Lh}{k}-\frac{Phx}{k^2}$$

$$W_T=\int\frac{T_x^2}{2A_1E}\cdot dx$$

$$\frac{d\,W_T}{d\,H}=\int\frac{T_x}{A_1E}\cdot\frac{d\,T_x}{d\,H}\cdot dx=\int_0^k\frac{T_x}{A_1E}\cdot\frac{m}{k}\cdot dx$$
$$=\int_0^k\frac{1}{A_1E}\left(\frac{Hm^2+V_Lhm}{k^2}-\frac{Phmx}{k^3}\right)dx=\frac{1}{A_1E}\left(\frac{Hm^2+V_Lhm}{k}-\frac{Phm}{2k}\right)$$

$$S_x=H\sin\theta-V_L\cos\theta-\frac{P}{k}x\cos\theta=\frac{Hh-V_Lm}{k}-\frac{Pmx}{k^2}$$

$$W_S=\int\frac{S_x^2}{2\alpha_1E_3}\cdot dx$$

$$\frac{dW_s}{dH}=\int\frac{S_x}{\alpha_1 E_s}\cdot\frac{dS_x}{dH}\cdot dx=\int_0^k\frac{S_x}{\alpha_1 E_s}\cdot\frac{h}{k}\cdot dx$$

$$=\int_0^k\frac{1}{\alpha_1 E_s}\left(\frac{Hh^2-V_1hm}{k^2}-\frac{Phmx}{k^3}\right)dx=\frac{1}{\alpha_1 E_s}\left(\frac{Hh^2-V_1hm}{k}-\frac{Phm}{2k}\right)$$

$$B_x=(V_1\cos\theta-H\sin\theta)x-\frac{P}{k}\cos\theta\frac{x^2}{2}=\frac{(V_1m-Hh)}{k}x-\frac{Pm}{2k^2}\cdot x^2$$

$$W_B=\int\frac{B_x^2}{2EI_1}\cdot dx$$

$$\frac{dW_B}{dH}=\int\frac{B_x}{EI_1}\cdot\frac{dB_x}{dH}\cdot dx=\int_0^k\frac{B_x}{EI_1}\cdot\left(-\frac{hx}{k}\right)\cdot dx$$

$$=\int_0^k\frac{1}{EI_1}\left[-\frac{h}{k^2}(V_1m-Hh)x^2+\frac{Phm}{2k^3}\cdot x^3\right]dx=-\frac{hk}{EI_1}\left[\frac{(V_1m-Hh)}{3}-\frac{Pm}{8}\right]$$

Right leg (measuring x to the left up the incline):

$$V_R=\frac{P}{2}$$

$$\frac{dW_T}{dH}=\frac{Hm^2+V_Rhm}{A_2Ek}$$

$$\frac{dW_S}{dH}=\frac{Hh^2-V_Rhm}{\alpha_2E_sk}$$

$$\frac{dW_B}{dH}=-\frac{hk}{3EI_2}(V_Rm-Hh)$$

Top (measuring x horizontally to the left):

$$T_3=H$$

$$W_T=\int_0^{l-2m}\frac{T_3^2}{2A_3E}\cdot dx=\frac{T_3^2(l-2m)}{2A_3E}=\frac{H^2(l-2m)}{2A_3E}$$

$$\frac{dW_T}{dH}=\frac{H(l-2m)}{A_3E}$$

$$S_x=V_R$$

$$W_S=\int_0^{l-2m}\frac{S_x^2}{2\alpha_3E_s}\cdot dx=\frac{S_x^2(l-2m)}{2\alpha_3E_s}=\frac{V_R^2(l-2m)}{2\alpha_3E_s}$$

$$\frac{dW_S}{dH}=0$$

$$B_x=V_Rx-Hh$$

$$W_B=\int\frac{B_x^2}{2EI_3}\cdot dx$$

$$\frac{dW_B}{dH} = \int \frac{B_x}{EI_3}\cdot\frac{dB_x}{dH}\cdot dx = \int_m^{l-m} \frac{B_x}{EI_3}(-h)\,dx = \int_m^{l-m} -\frac{h}{EI_3}(V_R x - Hh)\,dx$$

$$= \frac{h}{EI_3}\left[\frac{V_R m^2}{2} - Hhm - \frac{V_R(l-m)^2}{2} + Hh(l-m)\right]$$

$$= \frac{h}{EI_3}\left[-\frac{V_R l}{2}(l-2m) + Hh(l-2m)\right]$$

$$= \frac{h(l-2m)}{EI_3}\left[-\frac{V_R l}{2} + Hh\right]$$

Totalling and putting $dW/dH = 0$ we have

$$H\left[\frac{m^2}{Ek}\left(\frac{1}{A_1}+\frac{1}{A_2}\right)+\frac{(l-2m)}{A_3E}+\frac{h^2}{E_s k}\left(\frac{1}{\alpha_1}+\frac{1}{\alpha_2}\right)+\frac{h^2 k}{3E}\left(\frac{1}{I_1}+\frac{1}{I_2}\right)+\frac{h^2(l-2m)}{EI_3}\right]$$

$$= -P\left[\frac{hm}{Ek}\left(\frac{1-\frac{m}{2l}-\frac{1}{2}}{A_1}+\frac{\frac{m}{2l}}{A_2}\right)-\frac{hm}{E_s k}\left(\frac{1-\frac{m}{2l}+\frac{1}{2}}{\alpha_1}+\frac{\frac{m}{2l}}{\alpha_2}\right)\right.$$

$$\left.-\frac{hkm}{E}\left(\frac{\frac{1}{3}-\frac{m}{6l}-\frac{1}{8}}{I_1}+\frac{\frac{m}{6l}}{I_2}\right)-\frac{h(l-2m)m}{4EI_3}\right]$$

$$= -P\left[\frac{hm}{2Ekl}\left(\frac{l-m}{A_1}+\frac{m}{A_2}\right)-\frac{hm}{2E_s kl}\left(\frac{3l-m}{\alpha_1}+\frac{m}{\alpha_2}\right)\right.$$

$$\left.-\frac{hkm}{24El}\left(\frac{5l-4m}{I_1}+\frac{4m}{I_2}\right)-\frac{hm(l-2m)}{4EI_3}\right]$$

For an approximate formula we can disregard the effect of thrust and shear, and also assume the section to be uniform throughout. Thus:—

$$H = P\left[\frac{\frac{hm}{24}\{5k+6(l-2m)\}}{h^2\{\frac{2}{3}k+(l-2m)\}}\right] = \frac{Pm}{24h}\left[\frac{5k+6(l-2m)}{\frac{2}{3}k+(l-2m)}\right]$$

CASE 4.—FIXED ENDS.

Where the ends are built in the Theorem of Least Work affords the means of solution in that the first derivative of the internal work with respect to any load is equal to the displacement of that load in its own direction; while the first derivative of the internal work with respect to the moment of any external couple is equal to the angular displacement of the member at the point of application of that moment. As a rule, when ends are built in they are fixed so securely as to warrant the displacement being taken at zero; and, in consequence the differential co-efficients of the total work can be taken in respect to (1) VL, (2) HL, and (3) BL, equating each to zero, which, with the other three statical conditions that the sum shall be equal to zero,

respectively of (4) the vertical forces, (5) the horizontal forces, and (6) the moments, afford the six equations necessary to solve the six unknowns, consisting of three at each end, namely, the vertical and horizontal forces, and the fixing moment.

APPENDIX II.

DIMENSIONS OF RIVET HEADS.

By W. G. Perkins.

The greatest stress to which the heads will be liable will be that due to cooling of the shanks. This will not exceed 13 tons per square inch, as that is the limit of elasticity of rivet steel.

If d = diameter of shank in inches
D = diameter of head in inches
h = height of head at a point in continuation of the line of shank

Then the stress due to cooling $= d^2 . \frac{\pi}{4} . 13$ tons

The shearing area in the head $= \pi . d . h$
This shearing area at 5.5 tons per square inch must be sufficient to resist the stress due to cooling,

Therefore $5\frac{1}{2} . \pi . d . h = 13 . \frac{\pi}{4} . d^2$

Hence $h = 13 . \frac{\pi}{4} . d^2 \div 5\frac{1}{2} . \pi . d = \frac{13 . d}{22}$ say $0 \cdot 6\ d$ (1)

In cooling the rivet must not compress the plates or its head to a greater extent than 11 tons per square inch, and to this the underside of the head must be adapted.

Therefore $11 . \frac{\pi}{4} . (D^2 - d^2) = 13 . \frac{\pi}{4} . d^2$

$$D^2 - d^2 = \frac{13 . d^2}{11}$$

$$D^2 = \frac{24 . d^2}{11} = 2 \cdot 2\ d^2 \text{ nearly}$$

Whence:

$$D = \text{say } 1 \cdot 5\ d \qquad (2)$$

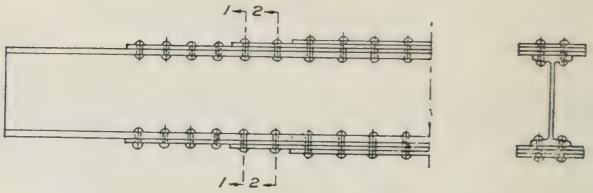
In practice the foregoing dimensions are usually

$h = 0 \cdot 75\ d$ (sometimes rather more)
$D = 1 \cdot 3\ d + \frac{1}{8}''$

APPENDIX III.

NOTES ON SHEAR ON RIVETS IN COMPOUND GIRDERS.

By H. Kempton Dyson.



Half Elevation. Cross Section.

Fig. 1.

At Section 1 there is a resistance moment R_1.

At Section 2 there is a resistance moment R_2.

The difference between R_1 and R_2 is due to the extra resistance contributed by the rivets in the length 1—2.

Let us take this as the distance between consecutive rivets, and assume that the rivets are not staggered (which, by the way, generally only increases the stresses in the flanges by reason of causing eccentric thrust or pull therein), but number n (i.e., 2, 4, 6, or more) across the width.

Then we are able to resist a change of bending moment in the pitch distance p equal to the change in resistance.

Referring to Fig. 2:—

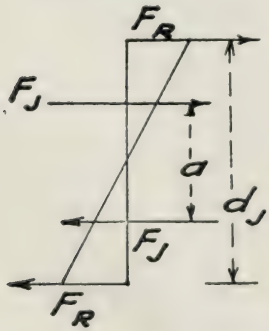


Fig. 2.

B = Bending Moment

S = Mean Shearing force in distance p

F_r = Force taken by Rivets

F_j = Force taken by Joist

d_j = Depth of Joist (i.e., arm of rivets)

a = Arm of Section

Now $F_j \, a = M_j$ = Modulus of Section of Joist

and $F_r \, d_j = M_r$ = Modulus of Section of rivets

The change in resistance moment between sections 1 and 2 being due to the rivets, we have:—

$$B_2 - B_1 = R_1 - R_2 = F_r \, d_j = n \, f_r \, d_j$$

Where f_r is the allowable force of resistance of one rivet of the size used, and n is the number of rivets across the breadth of the girder.

$$\text{But } B_2 - B_1 = S \, p$$

$$\text{Therefore } p = \frac{n \, f_r \, d_j}{S} \text{ and } S = \frac{n \, f_r \, d_j}{p}$$

In computing the position where plates may be stopped it will be noted that the resistance moment of the joist is reduced by the formation of rivet holes. It could be argued that, as the holes are filled up by the rivets, those on the compression side may be disregarded, but it is thought best to neglect them as this is not always so, and at the first rivets there is an insufficient number to be safe in taking an average, while the calculation is also simplified by the omission.

The use of a number of plates, and the use of rivets of a closer pitch near the middle of the girder, do not render the foregoing method inappropriate. The rivets should not be called upon to do more than carry the increment of bending moment at every part of the girder, and, if they do not do so, there is a lesser factor of safety against the yield point being exceeded. If the yield point be exceeded the rivets would continue to yield every time the girder was loaded to the same extent, until eventually they must fail.

APPENDIX IV.

DESIGN OF SPLICE PLATES.

By W. G. Perkins and E. Fiander Etchells.

As regards the question of the formula to be used in the design of slice plates for butted joints in pillars, neglecting shear, the stress induced in a joint will be the algebraic sum of the tensions and compressions caused by:—

(a) The wind pressure acting on the panels of masonry between the vertical pillars and the horizontal beams.

(b) The wind pressure tending to overturn the building about the leeward pillars.

(c) The bending moment induced by the eccentricity of the loads imposed upon the pillars by the beams.

(d) The bending moment induced by any system of wind bracing adopted, or, if there is no such bracing, by the stiffness of the joints in resisting the racking action of the wind.

With regard to (a) the panels of masonry may act either as slabs supported or fixed at four edges, or as vertical beams supported at the top and bottom by the horizontal metal beams, or as horizontal beams supported at the ends by the pillars, or by some combination of these ways.

The greatest effect upon the splice plates would be produced by the slabs acting as horizontal beams and the maximum bending moment would be approximately:—

$\frac{P H}{2}$ where P = one-half the total wind pressure acting upon two adjoining panels in a storey

H = the height of such storey

With regard to (b) tension on the windward side and compression on the leeward side would be induced in the pillars equal to half the total wind pressure on two adjoining bays multiplied by two-thirds the height of the building, divided by a factor depending upon the number of pillars appearing upon a transverse section.

If the pillars were evenly spaced across the building, before it would overturn about its leeward edge the tensions incited would be proportional to the squares of their distances from the leeward side.

Let P = the total wind pressure on a bay, calculated in accordance with the Act.

H = the height of the building in feet.

W = the width of one bay in feet.

N = the number of pillars in a transverse section.

T = the tension incited in the pillar first from the leeward side.

Then $T = \frac{2 P H}{3 W (1+2^2+3^2+\ldots(N-1)^2}$

and 2 T will be the tension on the 2nd pillar from the leeward side

3 T will be the tension on the 3rd pillar from the leeward side

and so on.

With regard to (c), the eccentricity in a symmetrical section may vary from $\frac{d}{2}$ to $\frac{l}{4}+\frac{d}{2}$; where d is the diameter of the pillar, and l is the length of the span of the beam, and depends upon the amount of fixity obtained in making the connection of the beam to the pillar; the greater the fixity the greater the eccentricity.

The bending moment will be further influenced by the ratio of the inertia moment of the beam to that of the pillar and by the degree of continuity in the pillar. As the actual degree of fixity cannot be accurately estimated, any consequential computation would be very approximate in respect of actual cases as met in practice.

The assumption commonly made is that the pillar is free at its upper end, and that the load is simply supported on the side bracket, causing a bending moment of P e; where P is the pressure load or reaction of the bracket, and e is the eccentricity, generally taken at one half the diameter of the pillar.

As to (d), the stresses induced will vary according to the system of bracing used, i.e., whether sway, knee, portal or deep beam. If no such system is used the effect of the wind must be resisted by the stiffness of the connections, and might cause a bending moment as high as $\frac{P}{N} \times \frac{H}{2}$

P being the pressure caused by the total wind pressure above the joint in question.
H the height of the storey.
N the number of pillars in a transverse section.

It appears to be unlikely that any tension can be induced in the splice plates.

As the joint is a butted one the compression will be taken up by the body of the pillar, and its effect on the splice plate need not be considered.

It is thought that the effect of the wind under head (b) can be neglected in all ordinary cases of several storied massive buildings. For instance, assume a building 80 ft. in height, with two storeys in the roof, of a total height of 100 ft.

If the spacing of the pillars = S = S_1 feet, the pressure would be (30 pds/ft.^{2}) $\times \left(\frac{2}{3}\right) \times$ (100 ft.) $\times$ S_1 ft. = 2,000 S_1 pds.

The overturning moment will be:—

$$(2{,}000\ S_1 \text{ pds.}) \times \left(\frac{2}{3}\right) \times 100 \text{ ft.} = \left(\frac{1}{3}\right) \times 400{,}000\ S_1 \text{ pds. ft.}$$

If the building were only one quarter of its height in width, i.e., 25 ft., the maximum tension on the windward side would be:— $\frac{1}{3} \times \frac{400{,}000\ S_1 \text{ pds. ft.}}{25 \text{ ft.}} = 5{,}333\ S_1$ pds.

Assuming the height of the storeys to be 10 ft., the weight of the floor and roof slabs alone would, taking steel and concrete 5 in. thick at 50 lb. per sq. ft., be

$$10 \text{ ft.} \times S_1 \text{ ft.} \times 50 \text{ (pds./ft.}^2\text{)} \times \frac{25}{2} \text{ ft.} = 6{,}250\ S_1 \text{ pd. ft.}$$

This leaves us with only (c) to consider. Accepting the formula W e, then the tension in the outer splice plate will be $\frac{P\ e}{d_s}$ where d_s is the distance from centre to centre of the splice plates. The tension in the splice plates is approximately $\frac{P\ e}{d} = \frac{P}{2}$; since $e = \frac{d}{2}$; where d is the diameter of the pillar, measured in the direction of e. Figured in this way the joint will be easily designed and possess an ample margin of strength. The shear will be $\frac{P}{N}$ and should be provided for in web splice plates.

It will, of course, be recognised that in the case of one-storey buildings the stresses due to wind will often exceed 25 per cent. of the stresses due to dead load, and special calculations of wind pressure will be necessary in such cases.

EXTRAORDINARY GENERAL MEETING.

An Extraordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, London, S.W.1, on Thursday, October 19, 1922, at 6 o'clock p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., A.M.I.Mech.E., Hon. A.R.I.B.A., &c.

The Minutes of the previous Extraordinary General Meeting were read, confirmed and signed.

The Chairman having addressed the meeting, Mr. F. E. WENTWORTH-SHEILDS proposed, and Professor HENRY ADAMS seconded, the following resolution :—

> " That this meeting confirms the resolution duly passed at the previous Extraordinary General Meeting held on September 28, 1922: That the title of the Company be changed to THE INSTITUTION OF STRUCTURAL ENGINEERS."

After debate the Chairman put the resolution to the meeting and declared it to be carried *nem. con.*

Mr. HENRY S. JARDINE proposed, and Mr. ALFRED BARNES seconded, the following resolution :—

> " That this Meeting confirms the resolution duly passed at the previous Extraordinary General Meeting held on September 28, 1922.
>
> That Article 3 of the Memorandum of Association of the Company be altered by omitting the several objects numbered 1 to 25 both inclusive therein, and by substituting the following objects in lieu thereof, namely :—
>
> (a) To promote the science and art of Structural Engineering in any or all of its branches.
>
> (b) To enable Structural Engineers to meet and correspond and to publish or communicate information relative to Structural Engineering, including the constitution, properties and use of materials.
>
> (c) Subject to the provisions of Section 19 of the Companies (Consolidation) Act, 1908, in any lawful manner to acquire, hold, and/or dispose of for the purposes of the Institution real and personal property, and any rights or privileges necessary or convenient for the said purposes, and subject as aforesaid to exercise in respect of property of any nature, all the rights and powers of an absolute owner of property of the nature.
>
> (d) To do all such other lawful things as are incidental or conducive to the attainment of the above objects.

After debate the Chairman put this resolution to the meeting and declared it to be carried *nem. con.*

Mr. A. SPENCER proposed and Mr. GOWER PIMM seconded the following resolution :—

> "That this meeting confirms the resolution duly passed at the previous Extraordinary General Meeting held on 28th September, 1922 : That the new Articles of Association already approved by this meeting, and for the purpose of identification subscribed by the Chairman thereof, be and the same are hereby approved, and that such Articles be and they are hereby adopted as the Articles of the Company in substitution for and to the exclusion of all the existing Articles thereof."

The Chairman put the resolution to the meeting and declared it to be carried *nem. con.*

A vote of thanks to the Council for all their work in bringing matters to the present successful conclusion, proposed by Mr. ALAN GRAHAM and seconded by Mr. CAMPBELL, was carried with acclamation.

A further vote of thanks to the Secretary, Assistant Secretary and staff and the solicitors, Messrs. Field, Roscoe & Co., and a vote of thanks to the President for acting as Chairman of the meeting, were carried with acclamation.

The proceedings then terminated.

NOTICE

ADVANCE COPIES OF PAPERS.

Members requiring advance copies of papers read during the Session, 1922-23, are asked to remit the fee of five shillings to the Secretary at their earliest convenience.

THE JOURNAL.

From and including January, 1923, the *Journal* will be enlarged to quarto size, and will include matter of interest to members in addition to the usual matter now published.

The Hon. Editor of the *Journal* is Mr. H. Kempton Dyson (Member), of 17, Warwick Square, S.W. 1, to whom communications should be sent.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. Incorporated 1909.

Vol. XII. *NOVEMBER, 1922.* No. 11.

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

CHANGE OF TITLE.

Due notice having been given, the petition of the Concrete Institute to change the title to THE INSTITUTION OF STRUCTURAL ENGINEERS, and to alter the objects contained in the Memorandum of Association, as passed and confirmed at the two Extraordinary General Meetings of members of the Institute held in September and October last, was heard before MR. JUSTICE EVE, sitting in the Chancery Courts, on Tuesday, 28th November, 1922. His Lordship was pleased to make the order granting the prayers contained in the Petition.

The formal licence of the Board of Trade is now all that is required, and it is expected that this will be received before the New Year.

The title of the *Journal* will remain unaltered in this and the December issue, to secure uniformity for the present volume.

The *Journal* for next year will bear the new title and, as previously announced, will be increased to quarto size, to include more matter of interest to members than has hitherto been published.

ORDINARY GENERAL MEETING IN JANUARY.

An Ordinary Meeting of the Institution will be held at Denison House, 296, Vauxhall Bridge Road, S.W.1., on Thursday, 25th January, 1923, at 7.30 p.m., when Mr. W. J. H. Leverton, F.S.Arc. (Member) will read a paper on "The Relations between the Architect and the Engineer."

THE BY-LAWS OF

The Institution of Structural Engineers.

FOUNDED 1908 AND INCORPORATED 1909 AS THE CONCRETE INSTITUTE. RECONSTITUTED 1922.

BY-LAWS AND REGULATIONS MADE BY THE COUNCIL OF THE INSTITUTION OF STRUCTURAL ENGINEERS PURSUANT TO ARTICLE 40 OF THE ARTICLES OF ASSOCIATION, AND BINDING UPON ALL HONORARY MEMBERS, HONORARY ASSOCIATES, MEMBERS, ASSOCIATES, ASSOCIATE-MEMBERS, LICENTIATES, GRADUATES AND STUDENTS OF THE INSTITUTION.

PRELIMINARY

The word "member" when used in these By-laws and printed in lower case shall signify a member in any class of the Institution.

The masculine gender includes the feminine.

The word "Institution" means "The Institution of Structural Engineers."

MEMBERSHIP.

CERTIFICATES OF MEMBERSHIP.

1. Every Member, Associate, and Associate-Member of the Institution shall, in accordance with Articles 22 and 23 of the Articles of Association, be entitled to receive a certificate of membership, which shall bear the common seal of the Institution, and be in such form as the Council may determine. Such certificates shall remain the property of the Institution, and in the event of a cessor of membership (except by death) shall be returnable on demand in accordance with Article 23 above mentioned. If a member be suspended under the provisions of Article 24 of the Articles of Association, his certificate shall be returnable to the Institution for custody during the period of suspension.

EXCHANGE OF CERTIFICATES.

2. Certificates of membership of the Institution under its former title of "The Concrete Institute" shall hold good as certificates of membership in the same class of the Institution of Structural Engineers.

A member (in any class) entitled to a certificate, elected prior to September, 1922, who desires to have a new certificate issued after that date may be supplied on payment of a fee of five shillings.

The new certificate shall bear the original date of election, but the signatories thereto will be the officials of the Institution for the time being empowered to sign certificates.

EXAMINATION CERTIFICATES.

3. A candidate who has passed any of the examinations of the Institution shall be entitled to receive a certificate, such certificate to be in such form as the Council may from time to time determine, and such examination certificate will be and will remain the property of the holder.

OFFICE-HOLDERS' CERTIFICATES.

4. The President and Vice-Presidents of the Institution shall on their election to office be entitled to receive a certificate of office, which shall be and remain the property of the holder.

SIGNATORIES TO CERTIFICATES.

5. The officials of the Institution empowered to sign certificates shall be the President, the Honorary Secretary, an Ordinary Member of Council, and the Secretary, for the time being of the Institution.

ADMISSION.

6. A candidate for admission to membership in any class of the Institution shall make an application on the official proposal form, and shall sign a declaration that he has read the Articles of Association and By-laws and is eligible and qualified thereunder, and that if elected he agrees to be bound by the Articles of Association and By-laws and any modifications thereof which may be made from time to time.

7. In the case of a candidate for membership (in any class) not being personally acquainted with the required number of members to propose him, he shall submit at least three letters of recommendation from responsible persons testifying to their personal knowledge of his qualifications and professional standing. In such a case the Council shall, if satisfied with his qualifications and with the status of the persons supporting his application, have power to nominate him.

TRANSFERS.

8. Associate-Members, Licentiates, Graduates and Students of the Institution desirous of being transferred to a higher grade of membership shall make application on the usual form.

Associate-Members, Licentiates, Graduates and Students on transfer to a higher class are not required to pay any additional Entrance Fee, but shall pay the difference in the current subscription between the respective classes.

TEMPORARY MEMBERSHIP.

9. The Council may at their discretion admit to temporary Membership or Associateship of the Institution, for a period not exceeding one year, any overseas Structural Engineer or Architect temporarily resident in Great Britain or Ireland whose admission the Council may consider to be in the interest of Structural Engineering or of the Institution, and such temporary Membership or Associateship may be extended from year to year by resolution of the Council. Such temporary Members or Associates shall have all the privileges of Membership or Associateship except the right to vote, and shall not be required to contribute to the funds of the Institution.

DISCIPLINE.

10. The procedure referred to in Article 24 of the Articles of Association shall be as follows:—

> Any complaint or charge under the Article referred to above preferred against a member (in any class) of the Institution, must be in writing, duly signed, and forwarded to the Secretary, who shall lay it before the Council at their next meeting. A complaint or charge may be made by any person, whether

a member of the Institution or not, and any member of the Council or the Secretary may at any time on his own initiative report to the Council any conduct on the part of any member which shall by any means have come to his notice, and which, in his opinion, ought to be taken into consideration by the Council. Such complaint or charge shall be entertained, considered and determined by the Council only, but the Council shall have power to appoint a Committee of not less than three of their own body to investigate it and report to them thereon.

Should the Council find *prima facie* ground for further proceedings, the Secretary shall send, in a registered letter, to the member against whom the complaint or charge is preferred (hereinafter termed " the respondent ") a statement of such complaint or charge, calling upon him to answer the complaint or charge within fourteen days of the date of such letter, either by letter or in person before a meeting of the Council. Should the respondent elect to answer the complaint or charge in person, or by Counsel, an opportunity shall be given to the person making the complaint or charge (hereinafter termed " the Complainant ") to attend the same meeting in person or by Counsel. In default of the respondent's compliance with the request to answer the complaint or charge, or if his explanation be in the judgment of the Council unsatisfactory, the Council shall have power to decree the reprimand, suspension for a period not exceeding twelve months, or expulsion of such member.

No member shall be suspended or expelled unless the Council so decide by a majority of at least two-thirds of those present and voting, and in any case by the vote of at least twelve of those present.

If the Council so decide to reprimand, suspend or expel a member the Secretary shall thereupon communicate the fact by registered letter to the member concerned. The fact of such reprimand, suspension or expulsion may, at the discretion of the Council, be published in the *Journal*, or any other publication of the Institution, and it shall in any case be communicated to the Chairmen and Honorary Secretaries of all Branches or Allied Societies of the Institution.

Provided always that should any facts be subsequently brought to the knowledge of the Council,

which, in their opinion, shall justify them in rescinding their previous decision in respect of such suspension or expulsion they shall have power to do so with the concurrence of at least the number of votes required for the suspension or expulsion as herein provided; and such decision shall be published and communicated in the same manner and in the same publications (if any) as was the decision to suspend or expel.

CONSTITUTION OF COUNCIL.

11. In accordance with Article 30 of the Articles of Association the business of the Institution shall be governed and managed by a Council consisting of a President, two Past-Presidents, four Vice-Presidents, one Delegate from each Branch or Allied Society of the Institution entitled to send a member to the Council, an Honorary Secretary, an Honorary Librarian, an Honorary Editor, an Honorary Curator, and the following twenty-one Ordinary Members of the Council:—Twelve Members of the Institution, of whom three shall be country members of the Institution; six Associates of the Institution, of whom three shall be country members of the Institution; and three Associate-Members of the Institution, one of whom shall be a country member of the Institution.

The two Past Presidents holding office as *ex-officio* members of the Council shall consist of the two junior Past Presidents the senior Past President automatically retiring as each new President is elected, but being eligible to seek re-election forthwith as an Ordinary Member of Council.

ELECTION OF COUNCIL.

12. Under Article 44 of the Articles of Association all those members of the Institution who, on the 27th day of April, 1922, formed the Council of the Institution shall retire, and shall be eligible for election or re-election, as the case may be. The procedure for conducting the first election of Council under these By-laws shall be as follows: Not later than 28 days previous to the holding of a Special General Meeting in May, 1923, the Council shall send to each member of the Institution entitled to vote a list of duly qualified persons whom they nominate for vacancies in the offices of President, Vice-Presidents, Honorary Secretary, Honorary Librarian, Honorary Editor, Honorary Curator, and Ordinary Members of Council. The list of persons so

nominated for the office of Ordinary Members of the Council shall be constituted as follows:— (a) It shall contain the names of at least eleven London Members of the Institution and five country Members of the Institution; (b) It shall contain the names of at least five London Associates of the Institution and five country Associates of the Institution; (c) It shall contain the names of at least four London Associate-Members of the Institution and three country Associate-Members of the Institution.

The election in 1923 shall be conducted in the manner hereinunder set forth, but the periods of service on the Council shall in 1923 be apportioned as follows:—The President, Vice-Presidents, Honorary Secretary, Honorary Librarian, Honorary Editor and Honorary Curator shall be elected for one year in accordance with Article 37 of the Articles of Association. Of the Ordinary Members of Council elected in 1923, the first three London Members, the first country Member, the first London Associate, the first country Associate and the first Associate-Member in order of voting, shall serve for three years; the second three London Members, the second country Member, the second London Associate, the second country Associate, and the second Associate-Member in order of voting shall serve for two years; and the third three London Members, the third country Member, the third London Associate, the third country Associate, and the third Associate-Member in order of voting shall serve for one year.

13. Every election of Ordinary Members of Council after 1923 shall be for three London Members, one country Member, one London Associate, one country Associate, and one Associate-Member, and those elected shall serve on the Council for three years in accordance with Article 44 of the Articles of Association.

Every election after 1923 shall be conducted in accordance with the Regulations laid down in By-law 12 *et seq.*, except that the number of nominations for the office of Ordinary Member of Council shall be in accordance with this By-law, and Article 45 of the Articles of Association.

14. After the issue of the Council's list any ten members qualified to vote may nominate in writing any other duly qualified person to fill any vacancy subject to their having previously obtained the consent in writing of such person to accept office if elected. The nomination paper accompanied by such consent must be delivered to the Secretary not later than the 21st day of April, and no nomination will be valid if the nominee or any of the nominators are under any liability to the Institution.

15. After May 1, a complete balloting list shall be prepared of those nominated for the Officers and Council, except the Permanent Officers, for the succeeding year, and the elections shall be conducted as follows:—

(i) The President, or, failing him, one of the Vice-Presidents, shall, from among the Members and Associates of the Institution, appoint not less than four Scrutineers, three of whom shall form a quorum.

(ii) The Secretary shall, at least seven days before the date of the Special General Meeting, forward a voting paper to every Member, Associate and Associate-Member of the Institution, who is not under any liability to the Institution, such voting papers to be in such form as the Council may from time to time direct.

(iii) The voting papers, duly sealed or otherwise fastened up as may be directed, shall be delivered by hand or by post prepaid to the Secretary at least two clear days before the date fixed for the Special General Meeting, and the Secretary shall deliver them unopened to the Scrutineers, by whom they shall be opened and examined.

(iv) As soon as the voting papers have been examined and the results of the election ascertained, the voting papers shall be placed in a sealed packet by the Scrutineers, and if no recount is demanded they shall be destroyed by the Secretary one month after the election is held.

(v) Any candidate for election to any office or to Ordinary Membership of the Council may within forty-eight hours of the announcement of the result of the election demand a recount of the voting papers. In the event of such recount being demanded, the President, or, failing him, one of the Vice-Presidents, shall forthwith appoint from among the Members and Associates of the Institution, four Scrutineers other than those already appointed, who shall, within seven days of the announcement of the result of the election, re-examine the voting papers. The decision of these Scrutineers upon the recount shall be delivered in like manner as the first decision, and shall be final. In the event of the recount showing no change from the first count, it shall be competent for the Council, if they think fit, to order the candidate who demanded the recount to pay the costs, if any, of such recount.

(vi) The Scrutineers shall make and sign a report in which they shall state the total number of voting papers received, the number rejected, and the grounds for rejection, the total number of votes in favour of each candidate, and the names of those duly elected, and shall hand such report to the Secretary not later than the day before the Special General Meeting.

(vii) In the event of there being an equality of votes, and if no recount be demanded, the Special General Meeting may give such casting vote or votes as may be necessary to remove the equality and complete the election. Such casting vote or votes shall be given by a show of hands, and no poll shall be demanded thereon.

LONDON AND COUNTRY MEMBERS.

16. A member (in any class) of the Institution resident within a twenty-five mile radius of Charing Cross shall be deemed to be a London member of the Institution. A member (in any class) of the Institution resident outside a twenty-five mile radius of Charing Cross shall be deemed to be a Country member of the Institution.

BRANCHES OF THE INSTITUTION.

17. Branches of the Institution may, in accordance with Article 73 of the Articles of Association, be formed at any time and for any place where deemed expedient by the Council for the time being, and each Branch shall be named after the country, colony, dependency, district or town in which it is formed, and where it shall have a permanent headquarters.

18. Branches of the Institution shall consist of Honorary Members, Honorary Associates, Members, Associates, Associate-Members, Licentiates, Graduates and Students of the Institution only. No Branch shall be formed or recognised by the Council unless it consists at the outset of at least twenty persons, including at least five Members of the Institution; and should the membership of any Branch fall below such number, the Council for the time being shall have power by resolution at any of its meetings, to declare such Branch to have ceased to exist, which resolution shall have the effect of putting an end to such Branch absolutely.

19. No Branch of the Institution shall be formed except with the approval of the Council for the time being, expressed by resolution to that effect on the written application of at least ten members of the Institution, five of whom shall be Members, who propose to promote such Branch.

20. Any member of the Institution shall be at liberty to give notice of objection to the formation of any Branch. Such notice of objection must be made in writing, duly signed, and delivered to the Secretary within one calendar month of the formation of the Branch.

Upon receipt of such notice of objection the Council shall submit the question of the formation of the Branch to the next General Meeting of the Institution provided that fourteen clear days' notice is given to the members of the submission of the question, otherwise at the next succeeding General Meeting, and the Council shall abide by the result of the decision made in General Meeting.

21. Branches of the Institution shall be ruled and governed by a Chairman, a local Honorary Secretary or local Honorary Secretaries of the Branch, and a Committee of at least five members; and three-fifths of the Committee, including the Chairman, shall be Members of the Institution.

22. The Committee of each Branch shall draw up the By-laws for such Branch, which must be submitted to the Council of the Institution for confirmation within three calendar months of the formation of the Branch. Any Branch shall, subject to confirmation by the Council of the Institution, have power at any time to alter, vary, rescind, or add to any such By-laws.

23. Branches shall be entitled to an annual grant of money from the Council of the Institution not exceeding one quarter of the total amount of the annual subscriptions actually paid by the members of such Branch. The working expenses of the Branch must be paid for out of this grant, and if they exceed the total amount of the grant, any additional expenses must be borne by the members of the Branch.

Such money grant shall be paid to the Branch on June 30th in each year.

No money grant will be made until a Branch has been formed for at least six months.

24. The local Honorary Secretary of a Branch shall furnish to the Secretary of the Institution on or before January 15th in each year, a balance-sheet of the accounts of such Branch to December 31st preceding, and such balance-sheet must bear the signatures as auditors of two members of the Branch, and must be countersigned as correct by the local Honorary Secretary.

25. A copy of each application form for membership in any class of the Institution made by a candidate resident

within the area governed by a Branch shall be forwarded to the Committee of the Branch, who shall send to the Council of the Institution within five weeks of the receipt thereof, an opinion as to the qualifications of the applicant, and any particulars concerning the candidate which the Committee of the Branch consider it desirable that the Council should have before them.

26. All members (in any class) of the Institution resident within the area governed by a Branch shall *ipso facto* be members of such Branch, unless such Branch charges a subscription, in which case it shall be at the option of each member to determine whether or not he will become a member of the Branch.

27. No Branch shall charge an Entrance Fee, nor shall any Branch require the submission of any form of application for membership other than the form required for membership of the Institution.

28. The imposition of an annual subscription to the Branch shall be at the discretion of the Committee of each Branch.

29. No Branch Rules or By-laws shall be valid, even after approval by the Council, if in the opinion of the Institution expressed by a resolution passed in General Meeting, such Rules or By-laws contravene any fundamental law of the Institution.

30. Every Branch consisting of more than fifty members whose subscriptions to the Institution are fully paid up shall be entitled to send the Chairman of such Branch as a Delegate to all meetings of the Council of the Institution. In the event of the death, incapacity, or refusal to act of the Chairman, the Committee of the Branch may appoint a member, being a member of the Committee of the Branch, to act in place of the Chairman as a Delegate to the Council

SECTIONS OF THE INSTITUTION.

31. Sections of the Institution may be formed from time to time in accordance with Article 73 of the Articles of Association by members of the Institution interested in a particular branch of the science and art of Structural Engineering.

32. No Section of the Institution shall be formed except with the approval of the Council for the time being, expressed by a resolution to that effect on the written application of twenty members of the Institution, fifteen of whom shall be Members, who propose to promote such Section.

33. The Council for the time being having approved the formation of a Section, the resolution to form such Section shall be placed before the general body of members of the Institution at the next succeeding General Meeting, and the Council shall abide by the decision of the members upon such resolution.

34. Sections of the Institution may consist of members (in any class) of the Institution, and of non-members of the Institution approved by the Committee of the Section, provided that if at any time there are less than one hundred members of the Institution in the membership of any Section, the Council for the time being shall have power by resolution at any of its meetings, to suspend the operation of such Section either *sine die*, or until the number of members of the Institution in the Section is augmented to reach one hundred.

35. Sections of the Institution shall be governed and managed by a Committee, consisting of a Chairman, Vice-Chairman, Honorary Secretary, and at least ten members of the Section, and three-fifths of the Committee, including the Chairman, shall be Members of the Institution.

36. The Committee of each Section shall draw up the By-laws of such Section, which must be submitted to the Council of the Institution for confirmation within three calendar months of the formation of such Section. Any Section shall, subject to confirmation by the Council of the Institution, have power at any time to alter, vary, rescind or add to any such By-laws.

37. Every Section consisting of more than two hundred and fifty members whose subscriptions to the Section are fully paid up, and who are not under any liability to the Institution, shall be entitled to send the Chairman of such Section as a Delegate to all meetings of the Council of the Institution. In the event of the death, incapacity or refusal to act of the Chairman, the Committee of the Section may appoint a member, being a member of the Committee of the Section, to act in place of the Chairman as a Delegate to the Council.

38. The headquarters of every Section shall be the headquarters of the Institution.

39. The working expenses of every Section shall, subject to approval by the Council, be borne by the Institution.

40. A member (in any class) of the Institution shall not be required to pay any Entrance Fee or Subscription to any Section, nor shall he be required to fill up any form of application for membership of a Section, other than a de-

claration that he will be bound by the By-laws of such Section and any modifications thereof.

41. Candidates for membership of Sections who are not members (in any class) of the Institution, shall comply with the By-laws of the Section to which they seek admission.

42. No Section shall consist of more than two classes of membership, one of which shall be a Studentship class. Members of the Institution below the rank of Associate-Member will belong to the lower class of membership in a Section, unless otherwise determined under the By-laws of such Section. Members of the Institution of and above the rank of Associate-Member shall in any case be entitled to the highest class of membership in any Section.

ALLIED SOCIETIES.

43. Any professional Society desirous of entering into alliance with the Institution shall in the first place make application to the Council, and shall forward a declaration signed by the President and Secretary of such Society that the application is made by a resolution of the members of such Society, passed in General Meeting by a majority of not less than two-thirds of those present and voting.

44. If the Council approve the application the matter shall be submitted to the General Body of members of the Institution by postal Ballot.

45. If the result of such postal Ballot prove favourable to the application, the result shall be communicated to the Society in question, which shall thereupon and thereafter be entitled to be described as "In alliance with the Institution of Structural Engineers."

46. Any five members of the Institution qualified to vote may at any time lodge a notice of objection to the continuance of an alliance with any allied Society, such notice to be signed by the five members lodging it, and delivered to the Secretary of the Institution, who will bring it to the notice of the Council. The Council shall within twenty-eight days of the receipt of such notice of objection take a further postal ballot of the members, in like manner as before. If the result prove unfavourable to the Society concerned, the Council of the Institution shall through the Secretary give such Society one month's notice to cease its description as an allied Society.

If the objection be not sustained by the General Body of members no further objection to the allied Society in

question shall be entertained for a period of twelve months; and it shall be competent for the Council at their discretion to order the members lodging the notice of objection to pay the costs of the postal Ballot.

DELEGATES ON THE COUNCIL.

47. Delegates from Branches or Sections on the Council of the Institution shall enjoy the same privileges as Ordinary Members of the Council, but they shall not be allowed to vote except on questions affecting their own Branch or Section, or the general question of Branches and Sections.

ALTERING BY-LAWS.

48. Proposals for any new By-law or the alteration, suspension, addition to, or repeal of any existing By-law of the Institution may be made to the Council in writing by any ten Members and/or Associates of the Institution. The adoption of any such proposal is at the discretion of the Council, subject to the provisions contained in Article 40 of the Articles of Association.

DISCREPANCIES.

49. In all cases where there appears to be a discrepancy between these By-laws and the Articles of Association of the Institution, the said Articles of Association shall govern.

PROCEEDINGS.

An Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, 7th September, 1922, at 5.30 p.m.

The Chair was taken by the President, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the Minutes of the previous Meeting, as published in the *Journal*, were taken as read, and were confirmed and signed.

The following members were elected in accordance with Article 8 of the Articles of Association:—

Members.

Cook, R. E., 2a, Cotsford Avenue, New Malden, Surrey; Cox, M. J., 9, Gloucester Street, S.W.1; Ellis, Francis Erlam, 61, Symonds Street, Auckland, New Zealand; Hanson, Arthur, M.R.San.I., Westfield Lodge, Bradford; Harrison, Percy, A.M.Inst.C.E., 36, Victoria Street, S.W.1; Healey, William E., M.S.A., M.R.San.I., Turner Street, Ramsgate; Johnson, William, M.I.Mech.E., Manchester Ship Canal Co., 41, Spring Gardens, Manchester; Mason, F. B., 104, Victoria Street, S.W.1; Mercer, J. A., A.M.Inst.C.E., 110, Cannon Street, E.C.4; Morris, Reginald S., A.M.Inst.E. (Australia), 8, Spring Street, Sydney, Australia; Parham, Arthur D., M.S.A., P.A.S.I., P.W.D., Colombo, Ceylon; Porter, J. B., Messrs. Homan & Rodgers, Ltd., 17, Gracechurch Street, E.C.; Rice, Sir Frederick Gill, 15, Stockwell Road, S.W.; Spencer, Albert S., Newton Heath Ironworks, Manchester; Stewart, W., Caixado Correio 1446, San Paulo, Brazil; Sykes, J., M.Inst.C.E., 296, Vauxhall Bridge Road, S.W.1; Wheeler, J. L., A.M.Inst.C.E., Craig's Court House, S.W.1; White, L. A., 9, Victoria Street, S.W.1; White, S. A., A.M.Inst.C.E., 9, Victoria Street, S.W.1; Woodfield, C. H., M.I.Mech.E., 6, Queen Anne's Gate, S.W.1.

Associate-Members.

*Anderson, John, 198, Mid Stocket Road, Aberdeen; Brooks, Douglas Clarles Morris, Port Said Engineering Works, Port Said, Egypt; *Grantham, C. J., 48, Leighton Road, Manchester; Penning, W. F., A.M.Inst.C.E., 8, Scarisbrick New Road, Southport; Marshall, Henry, 1, Rawdon Street, Calcutta; *Nixon, Joseph, 23, John Street, Workington.

*Passed Examination.

*Graduate.

Adams, Maurice E., 72, Queen Square, Bristol.

*Student.

Callaghan, T. F., 235, Wigan Road, Bolton.

*Elected by the Council.

Resignation.

The Secretary reported the acceptance by the Council of the resignation of A. T. Duguid, Member, of India.

Death.

Regret was expressed at the news of the death of N. F. Dennis, Member, of West Hartlepool.

The proceedings then terminated.

An Ordinary General Meeting of the Concrete Institute was held at Denison House, 296, Vauxhall Bridge Road, S.W.1, on Thursday, 16th November, 1922, at 7.30 p.m.

The Chair was taken by the President, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the Minutes of the Ordinary General Meeting of September 7th, 1922, and of the Extraordinary General Meeting of October 19th, 1922, as published in the *Journal* were taken as read, and were confirmed and signed.

The following members were elected in accordance with Article 14 of the (revised) Articles of Association:—

Members.

Abrams, Duff Andrew, Lewis, Institute, Chicago, Ill., U.S.A.; Burdett, Leonard Alphonso, 6, Queen Anne's Gate, S.W.1; *Corkill, F. M., A.M.Inst.C.E., P.W.D., Seremban, F.M.S.; Dailey, Arthur Benjamin, A.R.I.B.A., 97, Elspeth Road, S.W.11; Farquharson, John Frederic, 50, Garden Avenue, Mitcham; Fawcett, Philip Norrison, A.M.Inst.C.E., Lower Liao River Conservancy, Newchang, Manchuria; *Fox, Charles Horace, O.B.E., B.Sc., F.S.I., 7, Rawson Street, Halifax; Hagell, Frederic William, A.R.I.B.A., 32, Harberton Road, N.19; Hardinge, Henry Malcolm, M.S.E., c/o. Hong Kong & Shanghai Bank, Bombay; Hinks, George Rider, 1, Clifton Avenue, West Hartlepool; Jackson, Harry, A.M.Inst.C.E., M.R.San.I., 39, Newhall Street, Birmingham; Jones, Arnold Hugh Glenister, 6, Barmouth Road, Sheffield; Jones, Robert, Chief Inspector of Works, P.W.D., Lagos, Nigeria; McLean, William King, A.M.I.E. (Australia), 8, Spring Street, Sydney, Australia; MacPherson, James Alan, 14, Brockenhurst Gardens, Mill Hill, N.W.7; Morgan, Reginald Travers, A.M.Inst.C.E., A.M.I.Mech.E., City Building Surveyor's Office, Municipal Buildings, Liverpool; Perry, Capt. Reginald Clifford Sherrard (R.E., attached R.A.F.), The Bungalow, Halton Camp East, Bucks; *Tate, Newman, A.M.I.Mech.E., Sala de dibujo, Rio Tinto, Spain; Tigar, Leonard Aldridge, A.M.Inst.C.E., M.R.San.I., "Edgemont," Victoria Road, Prestatyn, N. Wales; Tollast, Walter Robinson, 6, Heathcote Road, St. Margaret's-on-Thames.

*Transferred from Associate-Membership.

Associate.

Tickner, Walter Ernest, 11, St. Margaret's Terrace, Plumstead Common, S.E.18.

Associate-Members.

*Bowker, Arthur, 52, Boulevard de la Citadelle, Gand, Belgium; Calley, David Ritchie, Kotree, Park Street, Bare, Morecombe;

*Davies, William Whitridge, 46, Springfield Road, Wallington; Hendry, Morrison, A.R.I.B.A., 3, Pitstruan Place, Aberdeen, *Pearson, Frederick James, 1, The Crescent, Anson Estate, Rusholme, M.R.; Plant, Charles William, Assistant Engineer, c/o. P.W.D. Secretariat, Rangoon, Burma; Smith, Edwin, A.R.I.B.A., P.A.S.I., 26, London Road, Neath; Timms, Ernest George, 39. Burnie Street, Waverley, Sydney, Australia; *Welch, Gordon, 15, Franconia Road, Clapham Park, S.W.; West, Edgar Ernest, A.N.Z.I.A., c/o. Collins & Harman, 81, Hereford Street, Christchurch, N.Z.

§Graduates.

*Averill, Charles William Stephen, B.Sc., 23, Trinity Grove, Bangoe, Hertford; Das, Nirmal Ananda, B.Sc., c/o. Grindlay & Co., 54, Parliament Street, S.W.1; *Dunn, Thomas, c/o. Messrs. Andrews, 324, Rumley Lane, Oldham; *Morgan, Ernest Eric, M.C., South Warnborough, Basingstoke; *Oliver, Edward Ambrose, 88, Golden Hillock Road, Birmingham; *Ward, Albert Frank, 1, Cambridge Villas, New Oscott, near Birmingham; Waterhouse, Norman, B.E. (Sydney), 78, Kensington Gardens Square, W.2.

*Passed Examination. §Elected by the Council.

Reinstatement to Membership.

Sands, Hubert, "Amhurst," Guibal Road, Lee, S.E.

Resignations.

The Secretary announced the acceptance by the Council of the following resignations :—C. H. Colson, Member, London; Prof. J. D. Cormack, Member, Glasgow; W. Crabtree, Member, Bradford; R. Cathcart, Member, U.S.A.; W. Ebdon, Member, York; W. Harling, Member, Brazil; E. J. Hill, Member, London; E. R. Marsland, Member, Persia; J. F. Plaister, Member, Seaford; Henry Puplett, Member, London; J. R. Osmond, Assoc.Member, London; F. Snowden, Member, France; H. J. Tingle, Member, London; G. P. Trentham, Member, Birmingham; C. O. Pugh-Jones, Student, Cardiff; D. W. Everitt, Student, Croydon.

Deaths.

Regret was expressed at the news of the deaths of the following members :—S. Kikkawa, Member, elected 1909, of Tokio, Japan; E. A. Royds, member, elected 1909, of South Africa; and S. A. Wisher, Associate, elected 1914, of London.

Mr. W. Marriott, M.Inst.C.E., then read a paper on "Concrete on British Railways," followed by a further paper on "Cements used by the Ancients."

A hearty vote of thanks was accorded to the Lecturer, and a discussion followed, to which Mr. Marriott replied.

The proceedings then terminated.

A LANCASHIRE AND CHESHIRE BRANCH.

At a highly successful meeting of local members and others interested, held in the Lecture Hall of the Engineering Department at Victoria University, Manchester, on Saturday, November 25, 1922, it was unanimously resolved to petition the Council for the formation of a Branch of the Institution of Structural Engineers, to be known as "The Lancashire and Cheshire Branch." The petition, which conforms to the regulations in the By-laws governing the formation of Branches, will be dealt with by the Council early in December.

After Mr. Albert Spencer had explained that he had only acted as convener of the Meeting, the Chair was taken by Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., President of the Institution, who, together with the Secretary (Captain Kiddy) had travelled up from London for the Meeting.

Mr. Etchells gave the Meeting a very clear exposition of what was meant by "Structural Engineering," pointing out that Civilian Engineering, as opposed to Military Engineering, was sub-divided into three distinct principal branches. One was the Mechanical Branch, dealing with Kinetics; this was represented by the Institution of Mechanical Engineers. The second was the Electrical Branch, dealing with Electricity, and this was represented by the Institution of Electrical Engineers; the third, and not the least important, was the Structural Branch, dealing with Statics. This last Branch had hitherto been represented only by the Concrete Institute, which had now changed its title to "The Institution of Structural Engineers," to denote more clearly its proper activities. He was sure that the Institution under its new title would in time rank equal with the two great Bodies he had just mentioned. Above all three Branches, in his opinion, was the Institution of Civil Engineers, which had within its ranks Engineers specialising in all the three Branches he had named. The three principal sectional Institutions were actually specialist organisations, and it might be that in time to come they would all three be federated with the parent Institution of Civil Engineers.

The President then explained that the Council was fully sensible that, whilst the routine work and general organisation of the Institution must be carried on from London as a universal centre, London could not claim, and did not wish to legislate for the Provinces, and machinery had therefore

been set up in the new Articles of Association and By-laws, for the establishment of self-governing Branches of the Institution, which would be represented by Delegates on the London, or main Council. The Council, however, did not wish to force Branches where these were not wanted by the the local Members, and the onus of inaugurating a Branch had therefore been thrown on the local members themselves, and in the present instance the meeting had been called together through the energy and initiative of a local Member —Mr. Albert Spencer.

On the other hand, wherever the desire to found a Branch was manifest amongst a group of local members, the main Council would be pleased to afford every assistance, and as in the present case, he himself as President, and the Secretary, were more than willing to attend in person at inaugural Meetings, in order to smooth over any difficulties in procedure and answer any questions.

The President having answered numerous questions addressed to him by those present, Mr. W. Galbraith proposed, and Mr. B. Taylor seconded, a resolution, " That this Meeting petition the Council to sanction the formation of a Lancashire and Cheshire Branch of the Institution of Structural Engineers." The resolution was carried unanimously, amid acclamation.

The Meeting then elected Mr. Albert S. Spencer as Chairman of the Branch for the ensuing twelve months, and Mr. Spencer took the Chair. The other officers and Committee were elected as follows:—*Local Honorary Secretary*, Mr. George Caddell (59, Hulme Hall Lane, Newton Heath, Manchester); *Committee*: Messrs. J. G. Evans, Major E. H. Heathcote, B. Taylor, F. E. Drury, H. H. Clapham, Professor Perrott, R. Travers Morgan, and Andrew Little.

A vote for dates for Meetings resulted in the second Wednesday in each month being chosen for the present: whilst it was understood that Meetings would be held in Liverpool and other large towns within the Branch area, from time to time.

Votes of thanks to the President, the Secretary, and the Chairman, concluded the Meeting.

REVIEW.

"Manual of Reinforced Concrete."—By Chas. F. Marsh, M.Inst.C.E., etc., and William Dunn, F.R.I.B.A., etc. Fourth Edition. Constable & Company, Ltd. Price 21s. nett.

The reappearance of an old friend is naturally welcome especially when increased bulk and usefulness are to be noted.

In the new edition of this excellent handbook, Mr. Dunn's analysis of the effects of brackets on columns, factors for doubly reinforced slabs with varying end conditions, and Professor Cain's treatment for pressures on retaining walls, constitute the chief additions to the previous issue. Minor alterations have also been made to bring the information up to date.

As a 5th edition will probably be required before long, it may be helpful to point out one or two misprints. In Table XXX. there seems something wrong with the co-efficients of friction for cement: in Table XXXII. the values for β and γ require checking: at the bottom of page 367 the load appears twice instead of once as a factor in the formula for arch thrust; on page 144 the symbols α and η should be replaced by μ and α respectively; a reference on page 45 to a past president of the Concrete Institute should unfortunately not have been made in the present tense.

W. A. G.

PORTLAND CEMENT

by

ROAD RAIL AND WATER

THERE is no difficulty in obtaining the Brands of Portland Cement, Lime, Whiting, Plaster, Chalk, etc., handled by the Cement Marketing Co., Ltd., for their works and depots are so wide-spread that prompt and regular dispatch to any part of the Kingdom is assured.

THE Company is anxious to give customers the benefit of its very extensive experience in the use of Portland Cement, and is always pleased to test and to give advice, free of charge, on aggregates which it is proposed to use for making concrete. Correspondence is welcomed on any points of difficulty.

Works, Wharves *and* Depots all over the United Kingdom.

— THE —

CEMENT MARKETING COMPANY, LIMITED

Selling and Distributing Organization of:—

The Associated Portland Cement Manufacturers, Ltd.
The British Portland Cement Manufacturers, Ltd.
Martin Earle & Co., Ltd.
The Wouldham Cement Co., Ltd.

8 LLOYD'S AVENUE, LONDON, E.C.3

Telegrams:
"PORTLAND, FEN, LONDON."

Telephone:
Avenue 5690 (Private Exchange.

GOVERNMENT SURPLUS STORES

EXPANDED METAL

Mesh 3 in.
Sheets 8 ft. by 4 ft.

Prices per sqr. Yard.

1,000 yds., 9d.
500 yds., 10d.
100 yds., 1/-

EXPANDED METAL

"B" pieces,
size 16 in. by 8 in.

Price per 100, 5,000 3/-;
2,000 4/-; 1,000 5/-

WRITE FOR LIST OF

BOW HUTS, STEEL TENTS, SHELTERS

(DEPT. I.C.),

GEORGE BLAY, New Malden, SURREY.

The Institution of Structural Engineers.

EXAMINATIONS, MAY, 1923.

Part I.—For GRADUATESHIP.

Part II.—For ASSOCIATE-MEMBERSHIP.

SYLLABUS ON APPLICATION TO THE SECRETARY OF THE INSTITUTION, 296, VAUXHALL BRIDGE ROAD, S.W.1.

Perfect Concrete

can only be made by properly selecting, proportioning and mixing the materials

CEMENT AGGREGATE & WATER

of which it is composed.

For the

BEST

PORTLAND CEMENT

ON THE MARKET

apply to

THE CEMENT MARKETING COMPANY, LTD.

Selling and distributing Organisation of

The Associated Portland Cement Manufacturers, Ltd.,
Martin Earle & Company, Ltd.,
The British Portland Cement Manufacturers, Ltd.,
The Wouldham Cement Company, Ltd.

8, LLOYDS AVENUE, LONDON, E.C.3.

Telegrams:
PORTLAND, FEN,
LONDON.

Telephone:
AVENUE 5690
(Private Exchange)

Brown & Tawse Ltd.,

OUR CATALOGUE contains exhaustive information concerning all forms of B. & T. Reinforcement, and is an invaluable book of reference to all Architects, Surveyors, Engineers, etc.

Copies will be sent upon request.

3, London Wall Buildings.
LONDON.
E.C.

REINFORCED CONCRETE

The best form of Monolithic Construction.

Rodbridge, Long Melford.

HOLLOWAY BROS. (London) LTD.
157, GROSVENOR ROAD, WESTMINSTER, S.W.1.

THE

JOURNAL

OF THE

CONCRETE INSTITUTE

AN INSTITUTION FOR STRUCTURAL ENGINEERS, ARCHITECTS, &c.

Founded 1908. **Incorporated 1909.**

Vol. XII. *DECEMBER, 1922.* **No. 12**

The Concrete Institute as a body does not accept any responsibility for the opinions expressed by individual Authors or Speakers.

CHANGE OF TITLE.

The change of title has now been finally approved, and the matter is concluded with the Authorities.

With the present number, the current Volume of the *Journal* also comes to at end, and from next month onwards the *Journal* will bear the new title, and will be increased to quarto size, with more literary matter than it has hitherto been found possible to include.

Binding cases for the present Volume can be obtained from the Secretary, at the Offices of the Institution, price 3s. 8d., post free. Back numbers of the current Volume (except August) are also available, price 1s. 2d. post free.

ANNUAL SUBSCRIPTIONS.

Annual subscriptions become due and payable on 1st January, 1923. They are :

Members and Associates	£3 3 0
Associate-Members...	2 2 0
Graduates	1 1 0
Students	0 10 6

CONCRETE ON BRITISH RAILWAYS

Together with some Notes on the Durability of the Natural Cements used by the Ancients

By W. MARRIOTT, M.Inst.C.E.

The writer feels much honoured by being asked to read a paper before this Institution as he recognises the fact that if we are to take our place at all in the front rank of concrete users (and we are very much behind at present), it will be largely through the operations of the Institution and the education it has given, and is giving, to our Engineers. He has, for many years, been an enthusiastic user of concrete and has never lost an opportunity of studying the question in all its bearings, making journeys abroad nearly every year to see some examples of foreign work.

BRIDGES.

He was first attracted to it through reading of Monier's experiments in Austria more than 40 years ago; in fact, many years ago when he commenced his Railway career, he was with a firm that had done a good deal of foreign work and who were among the first exponents of concrete in this country. We endured a great deal of opprobrium, and our concrete was not up to modern standards, or to present day knowledge, but as the proof of the pudding is in the eating, he is able to say that bridges built then have stood, and are standing to-day, in as sound or sounder condition than they were on the day they were constructed. There are not many Engineers who can say they have bridges in concrete nearly 50 years old on which not a penny has been spent on maintenance. In those days there was no steel in the concrete as reinforcement, as we now understand it, as the floor systems were generally formed by rolled iron joists which took all the weight and were embedded in concrete mainly for protection.

The only difficulty we have had with concrete has been that when we have had to pull down buildings, etc., for enlargement, so solid and sound were the structures, they were very difficult to remove, and the fact has also been noticed that the joists where they have been covered or embedded in concrete were as good as the day they were put in, and in many cases had even got the mill bloom still on them.

Since those days, whenever permissible, the writer has built scores of bridges of concrete, using, however, a brick facing as being cheaper than shuttering, and even going so far as to build in lime concrete, with excellent results, as was proved when one of these bridges near Cromer had to be taken down recently, the lias lime being as hard as a rock.

The use of pre-cast reinforced concrete in bridgework also offers many opportunities for economical construction.

It is quite possible, and the writer has done so, to make concrete girders for short spans complete and ready to lift into place.

In other cases, where the steel girders are encased in concrete, the floor system is economically made of arch blocks.

These arch blocks are pre-cast, and can be run into place as fast as men can handle them. They are then covered up with concrete *in situ*, and trains can be, and have been, run over within 24 hours. They form a bridge floor which is economical in first cost, and also in maintenance, especially where trains are continually running underneath, no temporary woodwork or centering is required, a good coat of lime white every now and then being all that is necessary.

There are many ways of designing bridge floors. Some Engineers prefer a flat slab. The writer considers that the arch is more scientific and economical. To give details of various possible methods of treatment would make this paper too lengthy: Suffice it to state what is being done.

He reinforced his first foundation about 1884, nearly 40 years ago, having some bridges to build on very bad ground, and it was necessary to make a kind of raft. Some 16 years ago he came to the conclusion that if concrete was to come to the fore, weight must be reduced, and the articles must be scientifically designed.

The writer was led to this largely by seeing an exhibition of concrete articles, which were so heavy and enormous, as not only to be wasteful in material, but costly to handle. He then formed the opinion that all articles where possible should be hollow, or be cored out in some way so as to reduce all surplus material, and made much as cast iron is in moulds by casting. So many had gone on the lines of simply reproducing in concrete the wooden article they were replacing, forgetting the adage that " Grandfather made his fortune because he made an old thing in a new way, or a new thing in an old way, or a new thing in a new way, never

the old thing in the old way, always the factor of novelty was one of the terms of his formula of success."

FENCE POSTS.

The writer started in some trepidation to design fence posts. These were tested with such satisfactory results that many miles have been put up with great success. They are considerably lighter than the standard type of some of our large railways, and consequently a good saving is effected. The Great Northern Railway have adopted a similar post with every success.

The next step was to make a straining post, and in this he was not successful at first. To get a straining post to stand the strain of eight wires tightly strained up meant a huge post, more than men could lift about, so he set himself to find a solution of this difficulty. It is not everyone, perhaps, who realises the severe strain that can be brought to bear upon an end straining post.

In trying many different types of reinforcement, it was found that the " Warren " type of bracing gave the best results for a minimum size post. This type of bracing was then adopted in various reinforcements, and tested beside other systems of reinforcement in the form of beams, etc., and in every case gave better results, even when only tied, and when welded the result was incomparably in favour of the " Warren " type, so much so, that after discussing the subject with some of his friends (and from experience), he decided to patent it.

SIGNAL POSTS.

The writer adopted the system for signal posts and erected the first post, 20 ft. high, in July, 1915. In March, 1916, we had the memorable blizzard, but the post, though unguyed, took no harm whatever. There were scores of trees just behind it that went down like nine-pins as did also telegraph posts for many miles. The system has been simply developed, and posts are now made up to 49 ft. long. Most of the principal railway companies have a large number, and two lines, at least, have adopted them as standard, which goes to strengthen the statement made by Mr. W. W. Grierson, in a paper before the Institution of Civil Engineers and at the Congress at Rome, that the posts were a success. He does not say he has not had any slight failures, but these were not due to any fault of the system generally, but to

faulty manufacture or transport and erection and general war difficulties, and we have now progressed so far as to make large bracket signal posts. One of the chief difficulties has been, and is still to-day, the education of the erectors, as in his experience men cannot expect a concrete post to stand the throwing about that a timber post will stand, or the deflection, but it is plain that many posts have had to undergo rough usage. There is also a certain amount of prejudice in favour of the wooden post, and it takes a good deal of demonstration to remove such prejudice. A great point in favour of concrete is, that once the post is up and sound, there will be no further expense on maintenance, and its life should be unlimited. One important factor is, that unless the concrete is made waterproof the consequent corrosion of the steelwork arising from the permeating moisture may cause the concrete covering of the metal, where too thin to burst off. The fault is well known to the expert. The writer has spent much time in experiments and has found a simple remedy. Cement concrete can be made impermeable by exercising due care in selecting the concrete aggregate, especially the sand, but as suitable aggregate is not always obtainable in actual practice, other means may be, therefore, at times necessary.

The reader is no doubt familiar with many waterproofing compounds under many different names which need not be particularised here, most of which the writer has tested, and some of which are absolutely valueless. Many of them claim to increase the strength and hardness of surface, but from practical experiment, although most of them bring about a waterproofing result, a great many of them decrease rather than increase the strength. He has also had independent analytical tests made which do not show that any claim to strengthen can be substantiated. The basis of all of them is either calcium or lime soap. The best method is doubtless to use a concrete which is waterproof in itself, and in that the reinforcement is protected by a sufficient thickness. In large structures it is possible to keep the reinforcement bars well inside, $1\frac{1}{2}$ in. to 2 in. away from the surface, but in small structures a covering of not more than $\frac{3}{4}$ in. to 1 in. is, in many cases, possible. He always aims at a minimum of 1 in. for posts or poles.

Travelling in France and motoring through the Alps the writer has had an opportunity of examining closely some of the power lines, and found even in that dry climate the

reinforcement, in many cases, on the surface, and even protruding, showing marks of rust which may ultimately bring the pole to grief.

For wrought work, like signal posts, which usually are painted, concrete can hold its own as to cost. It is not the same with telegraph poles, which we import so cheaply, and by creosting, the life is even from 30 to 35 years. To anyone who has compared telegraph poles in this country with those in France, especially in the south, where sometimes four poles are used where we should use one, it is not surprising that concrete has not made much headway in this country, nor is it anticipated that it will except for power lines.

Even in America the case is different. The deforestation, and the blizzards are turning attention to concrete poles. A gentleman who had resided many years in the State of New York was recently in the writer's office, and said that he had himself watched wooden poles go down one after another as far as he could see, during one of the blizzards. The question has been considered by the American Railroad Association, Telephone and Telegraph Superintendent's section. It seems that the deforestation and cost of maintenance and replacement of wooden pole lines compels them to search for a permanent substitute.

They arrive at the following conclusions that the high maintenance costs of steel or metal poles on account of frequent inspection and painting to prevent corrosion, places a ban on their extended use :—

That well made concrete poles will practically last for ever.

That the cost of wood and concrete poles are comparable when the life of the pole line is considered.

That the life of wooden pole lines depends greatly on the character of the soil and the insects contained in the soil.

They note that the large factors of safety allowed for wooden pole lines are not needed, due to the uniform behaviour of the materials.

They give a history of concrete poles in America, and finish by adding a recommended concrete specification to their report.

The writer has recently seen miles upon miles of such lines in France, Switzerland and Italy, and is quite convinced, from a durability point of view, that we shall, in the near future, adopt them for power lines. He has, however, never seen any finish abroad that is comparable to the finish of the poles we turn out.

The question of transport of concrete articles is a very serious one, and it is most necessary that design shall be carefully studied, so as to make the article as light as possible, consistent with its being strong enough.

The writer's system of reinforcement by its box-like form with no diagonals enables almost all articles to be made hollow without reducing the strength, which is of considerable advantage in handling and transport. To give some idea of how weight can be saved, he appends a few particulars.

A short concrete girder used in various positions for signal rod fixings in solid wood would weigh 27 lbs. per lin. ft., and in solid concrete 100 lbs., but as designed and made. 40 lbs. per lin. ft.

A signal post 27 ft. 6 in. over all in wood weighs 5½ cwts., if made of solid concrete 17 cwts., as made 15 cwts.

A 12 in. by 12 in. signal post leg (for iron bracket) 26 ft. 6 in. long in solid wood would weigh 11 cwts., in solid concrete 56 cwts., as designed and made 20 cwts.

Even taking every care in design into consideration, he was very early faced with the difficulty of the transport of large concrete power poles, and although abroad he had seen poles weighing from 1 to 2 tons in the most inaccessible situations, yet the cost of transport must have been considerable and difficulty great.

The writer, therefore, set himself to design a pole which would be easily transportable, and has achieved this by standardising 10 different tapering lengths each of 4 ft.; each piece being separately reinforced, and within certain limits. Out of these lengths, kept in stock, a pole of any desired strength can be selected and built into one pole by stringing them on main rods, at the place ot erection, the main rods engaging the individual reinforcements in the sections. This pole has been specially designed to meet the needs of electrical power transmission lines in undeveloped and trackless countries, so that native labour may be used to carry the pieces inland. He has been told by the highest experts that this pole has solved the difficulty of transport.

Another pole for light lines is the cruciform section, 6 in. by 6 in. or 5 in. by 5 in., is about the smallest square pole that can be made, the consequence is that the reinforcement is only about 3½ in. by 3½ in., and too near the neutral axis to give sufficient strength in many cases. The cruciform pole enables the reinforcement to be spread and

placed at such a distance from the neutral axis as to give a greater amount of strength for the same material, and has been specially designed for light telephone poles.

There is another very important point to be settled, and that is the question of the factor of safety. For wood we are agreed on a factor of safety of 10, and for steel 6, and in each case this factor of safety refers to the ultimate strength of the material. Now there is a movement on foot to put reinforced concrete on a different footing, that is, the ultimate strength of reinforced concrete shall be based on the first crack. This is most distinctly unfair, and, if adopted, will most certainly have a serious effect on its extended use since it will cause the cost to be prohibitive when compared with wood or steel for the same structures. The writer has been in touch with many experts who think as he does that a factor of safety of 3 for ultimate strength, would be sufficient for concrete power poles. It is, perhaps, necessary for the case of concrete roofs to be on a different basis.

The following tests made may be of interest :—

A round tubular pole made up of 4 ft. lengths, 24 ft. over all, was tested in a horizontal position, 5 ft. being firmly fixed and load applied 18 in. from top leverage 17 ft. 6 in. At 952 lbs. first joint crack appeared. At 1,960 lbs. the pole failed. The calculated safe load was 344 lbs. The factor of safety safe load to breaking load 5.7. Load at first crack 952 lbs. factor of safety safe load to first joint crack 2.76.

Safe load calculated for 28 tons steel 2,400 lbs. per sq. in. concrete (breaking strengths).

Again a test of a cruciform pole with a leverage of 15 ft. 8½ in. showed a factor of safety of 2.68 at first crack, failure at 807 lbs. showed a factor of safety of 6.

The whole question is an important one to manufacturers. Reinforced concrete, above all things, is a highly skilled product, and requires skilled supervision and skilled workmen, as well as first class material. The whole of the work, design, supervision, material, and workmanship, must be of the highest grade. A great deal of harm has been done to the profession by unscrupulous people putting cheap and badly designed products on the market with serious results.

In one case the writer received a report that one of his signal posts had failed, and when this came to be looked into it was found it did not refer to signal posts at all, but to fence posts which had been made and reinforced with old short pieces of iron bedsteads.

SLEEPERS.

With regard to reinforced concrete sleepers. The writer cannot say, at the present time, we have got a sleeper that will stand fast and heavy traffic, but he can only say that the sleeper he has designed is as good as any, if not, the best sleeper yet introduced. It has had a fairly good trial, and has proved satisfactory on slow lines, and he is very hopeful that in time we shall produce a sleeper that will be equal to all requirements, though not, he fears, as cheap as the wooden. A new design, not fully tested, has been got out.

The chief difficulty to be overcome in a satisfactory concrete sleeper, is the lack of elasticity, and there are many points of importance to be taken into consideration in designing sleepers, a few of which are:—

(1) The continuous repetition of impact to withstand which resilience is necessary, as well as first class concrete, excellent reinforcement, and the best workmanship.

(2) Constant impact consolidates the sleeper bed, so that there is a tendency for the sleeper to become unevenly bedded, and it may even ride on its middle point. The top then gets severe tension strains. If the track is trenched in the middle to avoid this the trench must not be made too wide, otherwise the effect will be reversed with equally unsatisfactory results, as experience has proved.

(3) The bearing area to ensure proper distribution of load requires consideration.

(4) The weight of the sleeper requires to be kept as low as possible which, therefore, limits the design and the strength.

(5) Maintenance men are somewhat prejudiced, and require to be trained in the management of this new kind of sleeper.

(6) Concrete sleepers are expensive, and will be so until a satisfactory type is produced, when they can be manufactured in bulk and the cost consequently reduced.

(7) Many of the best known sleepers are too lightly reinforced; others, notably the Italian, are over-reinforced, consequently cutting the concrete up into too small sections.

We have to go some distance yet before concrete sleepers can be said to be entirely satisfactory, but there are many entirely satisfactory replacements of timber in signal fittings, compensator frames, and general work, etc., where the carpenter can be completely eliminated; also in window

frames which is certainly one of the most economical items, being much cheaper than wood and most efficient, and equally suitable for wood or iron casements.

Another satisfactory product is Name Boards. The method of manufacture is somewhat novel, there being three colours of cement. The border is ordinary grey cement, the plate is of white cement, and the letters are black or red, or the colours of the board and lettering may be reversed. The letters are made separately, half the thickness being buried in the board and the other half standing out.

CEMENTS.

We have all, the writer trusts, ideals, and he hopes high ones. Perhaps he may be forgiven if he gives briefly what his ideals are.

(1) The very best cement. In this connection he would like to see a more rigid specification for reinforced concrete work. The British Standard specification is, he considers, too wide. He has had some very bad cement which just passed the tests, and could not, therefore, be thrown out, and yet the men in the shops, by simply handling the cement, have been able to tell him it was likely to prove unsatisfactory, and time proved they were right.

(2) A good mixing machine and clean washed aggregate of regular size, the latter being perfectly dry before being used. This is most essential if a perfect mix is to be obtained, and he hopes in best work to see two mixing machines used. The first to dry mix—the dry mix to be then delivered to a machine for water to be added. The correct amount of water is very important, and it is necessary to have rather more than the text-book amount to get a waterproof concrete, and tamp well into the moulding box round the reinforcement. The steel should be clean with a high elastic limit. Mechanical tampers or vibrators, carefully used, ensure thorough tamping and the exclusion of all air bubbles, making a dense homogeneous, and consequently permanent product irrespective of waterproofing compound.

(3) The scientific and practical training of the supervisory staff, as well as highly trained workmen. In some translating and sub-editing the writer has recently done of concrete papers for the International Railway Congress, great stress was laid on the training of all the staff by more than one writer.

It is, perhaps, permissible to enlarge a little on ideal No. 1. Although the question of cement is perhaps rather more in the province of the chemist than of the engineer, yet it occurs to him that we find all have a good deal still to learn on this subject if we study the works of our ancestors.

The writer will not go here into the relative merits of Kiln cement and Rotary, but just say that personally he agrees with many experienced Engineers who still hold that the old-fashioned Kiln cement is better and more reliable for reinforced concrete work than the newer process.

What we need now is cheaper cement, and if cement prices do not come down, and they have not in the same ratio as wood and steel, we shall have to turn to the cements the ancients used and see if an advanced chemical knowledge cannot turn them to good account.

Since penning the above, the writer finds that the suggestion he makes has been given effect to by Dr. L. R. Wentholt, Engineer-in-charge of the Meuse Lock on the Meuse-Waal Canal, who in these works has added pulverised trass to the cement, the theory being that Portland Cement contains too much lime (Ca O).

It is possible to make a chemical compound of the surplus of lime and silica (Si O_2) by adding trass. This compound of lime and silica does not dissolve in water. The trass cement is composed of one part cement and one-half part trass. By adding trass to Portland cement, the proportion between the main constituents, lime, alumina and silica may be altered as desired.

The advantage in Holland in using this instead of cement alone, is that in that country the price of trass is only half the price of cement.

A full description of the composition of this concrete is given in *The Engineer*, of September 15, 1922.

The writer would like before concluding to refer to some cements of nearly 2,000 years ago, as well as a modern case not well known.

During the Spring of this year he was a delegate to the Railway Congress at Rome, and, in company with other well known English Engineers, visited many of the ancient remains of that fascinating City.

What struck him, and also others, was that we evidently know comparatively little in these days of concrete, and especially cements, when we compare the results achieved

by the Romans in the buildings we saw. Is there an Architect, or Engineer, to-day that would put up a dome in concrete the size of the Pantheon in Rome? and yet that has stood for nearly 1,900 years.

What struck us so much was that with all the forces not only of nature, but of man, against the masonry and concrete, so much should be left standing. Man has wrought more destruction than time. As our official Guide truly said in the Baths of Caracalla, " It is man that destroys and not time."

One thing noticeable about the concrete and brickwork was, as regards the brickwork, that the joints and mortar (which is as hard as iron to-day) were as thick as the bricks, whereas the concrete was evidently placed very differently to our modern practice.

The mortar was evidently spread, and the stones arranged loosely in layers upon it, followed up by another layer of cement mortar, and so on, and we came to the conclusion that the Romans knew more about cement and concrete than we did, if after nearly 2,000 years so much remained. It is quite certain that this generation will not leave such an enduring evidence of civilisation behind.

It occurred to him that he would like to make a further research into the composition of some of these cements, but thought probably that this had already been done by the Italian Government. He, therefore, wrote to the management of the Italian State Railways asking if any researches had been made into the subject. After some delay they kindly sent him not only their article on the subject, but samples of ancient mortars from various buildings.

A general abbreviated summary of the remarks on ancient cements by the Experimental Institute of the Italian State Railways is as follows:—

" We know of many ancient examples of cements having a calcareous base to be met with in Egyptian monuments where in the joints of the stones which form the structure there are to be found thin layers of a cement of ordinary white lime and ground burnt clay; such a cement not hydraulic would evidently not be suitable except in a dry climate such as that of Egypt; nevertheless, the use of the cement in the Egyptian monuments is, on the whole, secondary, as it only forms a bed on which to lay the blocks in contrast to the Roman and Etruscan structures where the mortar mixed with broken stone becomes a structural material (concrete): such an Egyptian mortar is reported on in the paper giving

the mineralogical and micrographic description of a sample taken by Prof. Luiggi. Later calcareous mortars, of a composition even yet uncertain and little studied, came to be used in public buildings in ancient Greece, where, however, the structure was always covered according to the Egyptian custom, with hewn stone; it was only the Etruscans who began about 600 years before the present era, in the imposing structures which will remain, those of the Cloaca Maxima and in general in all the most ancient Roman sewers and the more ancient Roman bridges, the use of hydraulic cement having a base of fat lime and natural pozzuolana which constitutes the classic *Roman cement*; and which, mixed with pieces of stones, is the origin of the *Roman concrete.*

The composition and characteristics of this sort of building may be inferred from the references to them in the works of the ancient Roman authors: Marcus, Porcius Cato, Terence, Varro, Columella, and especially Vitruvius, give more or less complete particulars and information about such Roman buildings from which it appears that they were built of a mixture of fat lime, pozzuolana (Pulvis puteolanus which is obtained from Bozzuoli and the district round) and flints, the normal proportions of which were, according to Vitruvius, one of lime to two of pozzuolana.

Such a concrete, thoroughly hydraulic on account of the presence of the pozzuolana, came to be used by the Romans only in the hydraulic works of great importance exposed to great mechanical strain such as moles, jetties in the sea, foundations of bridges, &c.; in works which required less solidity they used instead of fat lime, gravel and broken bricks (coccio pisto) relying on the cementing action of burnt clay.

After the conquest of Gaul, near to Bonn, the Romans found a new kind of pozzuolana, "Trass," a kind of volcanic tufa, of which there were also considerable deposits in the valleys of the Brohl near Andernach and of the Eifel; the tufa, when ground with lime, gave an excellent hydraulic cement, which was largely used in their structures, and later came to be used on a large scale in Germany.

Where, however, the Romans could not find any hydraulic material, they used the ordinary mortar made of fat lime and sand obtained locally, taking care on this account to let the building set in the dry for more than 2 months by setting up suitable dams and drains. An example of a mortar of this sort is reported on in the description and micro photograph of a piece taken by Prof. Luiggi from the Roman bridge

of Septimus Severus over the Danube in Roumania.

The true Roman concretes having a base of lime and pozzuolana have given proof of remarkable strength, and also now after more than 2,000 years one may admire concrete, most of which has remained intact in the more important Roman monuments which remain to us; such as the bridges of Fabricius, Aemilius, Elius, Milvius, the Imperial quays, the arches of Claudius and Trajan at Ostia and Nero at Antium, &c.; the characteristic element which gives hydraulicity is the pozzuolana made up of an aggregate of minute fragments of lava paste with inclusion of mica, leucite, &c.; this pozzuolana is found in great abundance in the Italian volcanic zone (Bacoli, Pozzuoli in the neighbourhood of Vesuvius, the Roman Campagna near Cinima, &c.); chemically it is composed of complex double silicates of aluminium, iron, magnesium, sodium and potassium in very variable proportions; the hydraulicity seems to be due more to the special physical state in which it is found than to the constituent elements, in fact, pozzuolana exposed for a long time to the atmosphere deteriorates and becomes inactive. As to the internal action of the setting of the lime with the pozzuolana, opinions differ, and no theory has yet been formulated on which all can be agreed; the hypothesis most generally believed is that the silica and alumina in the pozzuolana combine with the lime to form double hydrated silicates of calcium, aluminium and iron which by crystallising cement the mass together; the essential difference between the Roman mortar and mortar made from modern cement is in the fact that while in the latter the unhydrated silicates and aluminates of calcium form hydrate silicates and aluminates of calcium in setting, in the pozzuolana mortar it is thought that more stable double silicates of calcium, aluminium and iron are produced, especially in the presence of sea water, this explains why pozzuolana concrete is more successful than modern Portland cement concrete in marine work."

Coming now to more recent times, there is a new cement which is not yet much known; hardly at all in England, and very little in France, called by the French "Ciment Fondu." Through the kindness of Monsieurs Quinquet and Ruffieux, the Chief Engineer and Assistant Engineer respectively of the P.L.M., the writer has had a report of which the following is a brief resume. Originally the subject of a Patent it is:—

A mixture of limestone and bauxite (aluminium ore

composed essentially of aluminium hydroxide with iron hydroxides, clays, &c.) fused with coke in a water-jacket.

At present it is manufactured in an electric furnace. The fused mass is cooled and broken into small pieces which are ground to powder in a ball mill. It agrees approximately with the following analysis :—

S_1O_2	... ...	about 10
Al_2O_3	... ...	,, 40
Fe_2O_3	... ...	,, 10
CaO	... ...	,, 40
		Per cent.

The following is an analysis made in the laboratory of the Ecole des Ponts et Chaussees :—

Silica in soluble form (or compounds) ...	11.85
Sand	0.65
Alumina	31.82
Ferric oxide	13.28
Lime	40.80
Magnesia	0.97
Sulphuric anhydride	0.11
Loss in heating ($H_2O + CO_2$, &c.)	0.05
Unidentified and lost elements	0.30
Alkali	0.30
Sulphur in the form of sulphide	0.34
Deduct : Oxygen in combination with sulphur	0.17
	100.00

Specific gravity, 3.12.

The chemical composition may, however, vary to some extent without affecting the quality of the cement. The quantity of lime added to the bauxite varies according to the quantity of silica contained in the latter. With red bauxites it may be only necessary to have :—

50 limestone
100 bauxite

with white bauxites

100 limestone
100 bauxite

These various mixtures fuse at about 1400° ; the fused cement is very fine ; it usually leaves only 4.50 per cent. of its weight on a sieve of 180 meshes to the inch. It has a high initial strength. Setting begins at 1½ hours and finishes 3½ hours.

The temperature in a gauging may rise as high as 150° during the process of setting. As it soon attains considerable

strength it was used during the war for mounting heavy guns.

It resists water containing sulphur. This cement has been subjected since 1916 to very severe tests in the lake at Sospel (Nice Coni line).

The waters, which feed this lake, are charged with sulphur. In the experimental blocks the fused cement resisted decomposition perfectly. After 3 years of testing fused cement has been used entirely. This cement is very dear, $2\frac{1}{2}$ times good Portland cement, but no doubt more will be heard of it.

This cement should be valuable in special work where sulphur is present and, no doubt, is the precursor of other likely discoveries which will aid reinforced concrete in the future, as no doubt many improvements will be made as soon as we as a nation lay aside our prejudices.

In conclusion, we do well to remember the well-known lines :—

" The heights by great men reached and kept,
Were not attained by sudden flight,
But they while their companions slept
Were toiling upward in the night."

APPENDIX I.

EXAMINATION OF EGYPTIAN MORTARS FROM THE PYRAMIDS.

Egyptian mortars—the more " healthy " samples, or rather those in which the mortar was best preserved, were taken from the wide joints between the stones at the north-east corner of the pyramid of Cheops, where the action of the very hot sun and the dry wind and sandstorms of the desert was least felt, but where on the other hand, the effects of the variation of temperature between the day and night were more felt, this latter having a great influence on the disintegration of the mortar.

From the examination of such samples, made with the diligent co-operation of the eminent Engineer, Mr. C. Segre, head of the Experimental Institute of the State Railway, it appears that the mortar was fine lime and burnt clay, probably broken bricks and brick dust; and from a test piece carefully cut from a piece of the mortar, the following results were obtained :—

Dimension of test cube.	...	...	1.8 cm.
Apparent specific gravity of the mortar	...		1.3
Crushing strength per cm^2	...	...	5.7 kg.

This mortar is probably similar to that which is still made in Egypt with pounded burnt clay, called homra, and is now used in subaqueous work of secondary importance and especially where it is necessary to put in large quantities of concrete in foundations for syphons, sluices and similar works. Usually 3 parts by volume of ground slaked lime are used with 2 of homra, and the resulting mortar is used to make concrete in the proportions of 1 of mortar to 2 of flints or broken bricks. If this concrete is put down in water it takes several days to set, but if it is laid and well rammed between partitions which are kept dry, and is left for a few hours until setting has begun, and the water is let in slowly so as to cover it, setting will be complete in less than 18 hours.

APPENDIX II.

Comparative Tests between Pozzuolana of Salone and Portland Cement.

Pozzuolana from the Quarry of Salone.

Weight in lbs. per cub. ft. { Soft 72 ; Hard 90

Three parts in weight of Pozzuolana to one part in weight of powdered lime mixed by hand.

		28 days	84 days	210 days	1 year	2 years	
Compression Test.	dry	381	422	398	636	829	lbs. per sq. in.
	under water	387	567	860	936	1,370	,,
Tensile Test.	dry	80	82	109	125	125	,,
	under water	254	166	252	284	315	,,

2½ parts in weight of Pozzuolana to one part in weight of Portland Cement.

		7 days	28 days	84 days	1 year	
Compression Test.	dry	321	628	600	1,188	lbs. per sq. in.
	under water	280	646	1,022	1,521	,,
Tensile Test	dry	85	87	78	154	,,
	under water	71	133	166	202	,,

NOTE.—

There are great variations in the test results of Pozzuolana according to the Quarry from which it has been taken.

PORTLAND CEMENT TEST.

Weight in lbs. per cub. ft. } 86 to 94.

	7 days	28 days	
Tensile Test	450	540	lbs. per sq. in. B.S. Specification.
	575	685	,, ,, ,, ,, Author's own averages.

Portland Cement mixed with Standard Leighton Buzzard Sand, 3 parts sand 1 part cement.

	7 days	28 days	
Tensile Test.	200	250	lbs. per sq. in. B.S. Specification.

Portland Cement mixed with Holt Sand, 3 parts sand 1 part cement.

	7 days	28 days	
Tensile Test	110	201	lbs. per sq. in. Author's own averages.

DISCUSSION.

Mr. W. J. H. LEVERTON, who proposed a vote of thanks to Mr. Marriott for his interesting paper, said it was very satisfactory to find a railway engineer talking of concrete, because, as a rule, railway engineers rather cold-shouldered it. Mr. Marriott had spoken of arches, and had said that arch blocks formed a bridge floor which was economical and satisfactory. It was always rather a mystery to him (Mr. Leverton) to find that engineers, especially concrete engineers, did not make more use of the arch, because concrete was essentially a material which had its main strength in compression. Of course, where they had tension they put steel in, but when they used an arch they simply used the compressive strain, and did not want steel. The author had referred to the fact that the dome of the Pantheon in Rome was still standing, after 1,900 years, and had said that most of the concrete erections of the present day would

not stand so long. Of course, there was the question of the thickness of the concrete. Mr. Marriott had not mentioned what the thickness was, but he (Mr. Leverton) expected that it was more than about the 7 inches which a modern engineer would lay, and that accounted for the long time that the old buildings had stood. The Roman engineer built for eternity, but the modern engineer had to build to cost; not only had he to consider cost, but also the space which those thick walls and arches would take, so that, with all due deference to Mr. Marriott, he thought it hardly fair to make a comparison between Roman concrete erections and those of the present day. Mr. Marriott had also referred to bricks. Of course, the Roman brick was over 10 inches, and the Romans built in a course of brick work, both vertical and horizontal. It was just possible that these brick courses were extended as expansion and contraction joints, so that if a crack occurred it followed the path of the brick work, and did not go straight across the slab. That might or might not be the case; very likely it was.

The PRESIDENT, referring to the point raised by Mr. Leverton as to the endurance of some of the old buildings, said that perhaps that endurance was because the Egyptians built for eternity, the Romans built for posterity, the modern builder has to build "to time," and at competitive prices too. These things made a difference.

Mr. D. B. BUTLER said he had very great pleasure in seconding the vote of thanks to Mr. Marriott for his very interesting paper. As one interested in concrete, it was very gratifying to him to see that so much progress had been made in the use of concrete in railway work, for posts, sleepers, and so on. In that respect we were far behind American practice, and we had a good deal to learn from American practice. The chief reason for the economy of concrete was that, once it was there, and properly made, it was practically everlasting; that was, of course, provided that the reinforcement was sufficiently protected to avoid rusting. His chief study for many years past had been cement. He agreed with the author when he had said that he would like to see a more rigid specification for Portland cement. The author had suggested that the British standard specification was too wide, but he (Mr. Butler) would rather say that it was too "weak." He had had, in the course of a year, very many samples submitted for testing purposes from all parts of the world, and they had demonstrated the fact that the British standard specifica-

tion was very much too weak to ensure a good average cement. His experience was that the English cements, and most of the foreign cements, developed a strength at least 50 per cent. above the minimum required by the British standard specification, and a great many cements developed a strength of 100 per cent. above that. Also, with regard to fineness of grinding, this was laid down in the British standard specification as 14 per cent., but nearly all the first class cements made in England and abroad were ground to leave a residue of about 5 per cent. on a 180-mesh sieve, so that the specification should be strengthened considerably. That would keep out a great many cements which at present only just squeezed through, and were very much weaker than a good average Portland cement. With regard to the statement that the men in the shops knew, by handling a cement, whether it was good or bad, he considered that a man must be clever if he could tell from its appearance. Some years ago he had heard of an old works foreman in a German works, who must have had a most particular palate, because he could always tell whether cement was good or bad by tasting. He (Mr. Butler) doubted whether that method was good enough. The author was rather in favour of cements produced by the old-fashioned bottle or intermittent kiln, as against those produced by the rotatory kiln process. The fact that at least 95 per cent. of the cement produced in England at the present time was produced by the modern rotatory process was an argument in favour of the quality of that product, and he believed that at present, since it was also a more economical method of production, the author would find some difficulty in getting cement produced by the old-fashioned bottle kiln method. As to the use of trass with Portland cement, that was recommended by Dr. Michaelis, the great German expert on cement, some years ago. Trass was a volcanic product, chiefly silica and alumina, the great point being that the silica was mainly soluble, and the theory was that the soluble silica in the trass combined with the lime in the cement which was set free when the cement set, so forming a stronger product. Also, it prevented the cement being attacked by sea water. Some years ago he had reported on trass for a German firm—he did not remember the proportions—but it certainly did give a better result than pure cement, especially in sea water. With regard to Ciment Fondu, he had had the opportunity of examining this. It was made in the South of

France, on a totally different principle from that on which Portland cement was made. In making Portland cement they had to operate it to such a degree that they stopped short of fusion of the raw material, but the Ciment Fondu was actually fused to slag, and the great difference was that it was very highly aluminous compared with Portland cement. It was black, and it had the great advantage of developing extremely high strength in a very short time. He had had samples submitted to him, one from the manufacturers, two years ago, and one from a user in India, recently, and the results of the two samples compared very closely. As to fineness of grinding, the residue on a 180-mesh sieve was only 2 per cent. The initial setting time was 1½ hours, and the final setting time 4 hours. As to strength, in one day, 24 hours after gauging, the neat briquette developed a tensile strength of 520 lbs. per square inch; at 2 days it was 800 lbs.; in three days, 900 lbs., and so on, but it did not seem to increase much after 28 days. A 3 to 1 sand-cement mortar developed a strength in one day of 387 lbs. per square inch; in two days, 447 lbs.; in three days, 468 lbs.; and in seven days, 500 lbs. Since the British standard specification stipulated only 250 lbs. per square inch for 28 days, it would be seen that this cement would pretty well double the strength of Portland cement in about two days. Another important point was that it seemed to have an exceptionally high resistance to crushing compared with the tensile strength. The usual ratio of crushing strength to tensile strength of Portland cement was, roughly, from 8 to 1 to 12 to 1. In this case, however, the ratio was 12 or 14 to 1. The analysis of the material showed it to contain about 10 per cent. silica, 40 per cent. alumina, 5 per cent. iron and only 42 per cent. of lime. Mr. Butler pointed out that he was not in any way interested in this cement; he was merely giving the figures for information. It was a new thing, and although, no doubt, it was expensive, it might be used successfully in many cases where quick strength was required, and it had been used, as the author had said, for mounting guns during the war.

Mr. H. E. Steinberg, after expressing his own thanks to the author, said that if it were possible he would like him to give further information with regard to the data on which the poles for electric power transmission lines were designed. The firm with which he was connected had had experience, in France, of the design of power poles for heights up to 17 metres, and in France the spacing between

the poles was generally considerably larger than was usual in this country. From the point of view of design, the factors which had to be taken into account were very many, and some of them were very difficult He believed that the Ministry of Transport was considering this matter, and that before long it would formulate instructions for the guidance of people designing these poles. There were problems that one could think of at once, such as windage on one side, the possibility of one or more cables breaking, and the pole being, in consequence, subjected to an unbalanced pull, and many others. It would be very interesting if the author could give some idea of his practice in regard to these matters. He was very much interested in Ciment Fondu. This was undoubtedly a material which, if it could be produced on a commercial basis, might have a most revolutionary effect on reinforced concrete construction. As a practical instance of what had been done, and what could be done, one of his co-Directors had told him that the firm had made a number of piles in Ciment Fondu, and in three days had driven them. To those who had had experience of making or driving ordinary cement piles, that was almost incredible. The firm had also used Ciment Fondu for lengthening piles, in cases where piles had been driven to a certain depth, and it had been found necessary to lengthen them in order to obtain proper penetration, and these lengthened piles had been driven two days after lengthening. Obviously, that opened up tremendous possibilities, and would enable many things to be done in reinforced concrete which at present have to be done some other way. He was sure that the Institution would take great pains to keep in touch with developments in connection with Ciment Fondu.

Mr. M. E. Yeatman said he had been specially interested in the author's reference to Ciment Fondu. It looked as though possibilities were being opened up on the cement and concrete side which were analogous to the possibilities introduced in connection with long bridge spans by high tensile steel alloys. That being the case, he was anxious to get further information, and Mr. Butler had supplied this to some extent, in connection with the compressive strength of the blocks. If he could have also the modulus of elasticity of the blocks in compression, it would be interesting as regards the theory of design, in spite of Mr. Dyson's new theories.

The President said that Ciment Fondu would undoubtedly have its uses, but there was a big lesson behind it all, namely, ceaseless progress, ceaseless change. As

soon as they got a product standardised by the Engineering Standards Committee, and were getting on nicely, a new material loomed on the horizon. As soon as Ciment Fondu was standardised, somebody would come along with another material. Some of them were getting old, and were getting dizzy with such rapid changes, but the Institution would live; those who came after the present members would probably be talking of Ciment Fondu as some ancient material, and he hoped the Institution would always open its doors to the discussion of all that was new and progressive in constructional matters.

Capt. F. J. Rouse, who said he had had the privilege of being an old pupil of Mr. Marriott, dealt with his personal knowledge of the use of pozzuolana in Greece. Greece possessed, in the island of Santorino, an island composed entirely of pozzuolana. This was a crater of an extinct volcano, divided into two sections, so that ships of almost limitless draught could get right alongside the solid cliffs of pozzuolana on the Western side. He had had an offer of pure pozzuolana at 7 drachmas per ton f.o.b. Santorino. There might be some difficulty in getting transport, and that would be a great expense, but the material was simply quarried at the tops of the cliffs and shot straight into the ships. The whole of the houses in Santorino were built entirely of pozzuolana and lime; the inhabitants had built them in exactly the same method for the last 2,500 years, and were still perfectly satisfied with them. Of course, 7 drachmas, at the standard rate of exchange, would be about 5s. 6d., but at the present rate it would probably be about 3½d. The price had gone up since he had received the offer, however. There was in Athens a works in which were made bricks of pozzuolana and lime, and an enormous quantity was turned out every day; they were used for building largely. Needless to say, the manufactory was in the hands of Germans, and had been put up entirely with German capital. In conclusion, he said he had some 20 kilogrammes of pozzuolana which had been brought from the island, and he would be glad to supply quantities of it to anyone desiring to make tests on a small scale. There was also about a ton in the London docks, sent at the request of a Greek firm in London, but of which delivery had never been taken.

Mr. Marriott, replying to the discussion, expressed his appreciation of the vote of thanks accorded him. The preparation of the paper was a labour of love to him. He was

rather glad to hear Mr. Leverton say that railway engineers were not usually in favour of concrete, and if the present audience knew the opprobrium he had had all his life they would be amused, but he had stuck to it, and was coming out on top. His Directors were beginning to believe in it, but it took time. As to the use of arches, he always went in for them when he could. In this country, however, the headways were limited. The thickness of the Pantheon haunches was about 6 or 7 ft. He was exceedingly interested in Mr. Butler's remarks. If such an authority as Mr. Butler was going to move in the direction of getting the British standard specification altered, we should be doing something. Generally speaking, if they got a cement which looked yellow and nasty they knew it was not going to turn out well. When he had said that experienced workmen could tell whether a cement would turn out to be good or bad he did not mean, of course, that he relied upon that, but it was very extraordinary what the ordinary working man knew. After handling materials all their lives, they knew them by instinct. As to the ordinary kiln and the rotatory cement, of course, he knew that that was a debatable point, but he also knew that one of the best firms in England were putting up more kilns, so that there was something in it. Mr. Steinberg had touched a very interesting point, and one which he (Mr. Marriott) had been studying for years. In his practice—and he had had some of the biggest electrical men in connection with him—he took the wind strain at the sides alone and did not consider the end load which would be caused by a broken cable, because if a cable broke it was bound to slip through the insulator quickly, and the load altered. Unfortunately, as Mr. Steinberg had pointed out, there was no fixed rule. Some people would say that they wanted a pole with a factor of safety of 6, and some wanted a pole with a factor of safety of 3, and the poles were simply designed for the weights, for the spans and for the factor of safety asked for. The subject was very big and very difficult, and it would take more time than he had at his disposal to answer it. He hoped that the Institution would do something to get the British standard specification for cement amended. He himself stuck to a particular brand for reinforced concrete, and there were plenty of good brands which could be depended upon. It was most important, if reinforced concrete was to come to the fore, that absolutely reliable cements must be available.

PROCEEDINGS.

An ORDINARY GENERAL MEETING of The Institution of Structural Engineers was held at Denison House, 296, Vauxhall Bridge Road, London, S.W. 1., on Thursday, 14th December 1922, at 7.30 p.m.

The Chair was taken by the PRESIDENT, Mr. E. Fiander Etchells, A.M.Inst.C.E., &c., and the Minutes of the previous Meeting, as published in the *Journal*, were taken as read, and were confirmed and signed.

The President announced that the change of title had now been formally approved, both in the High Court of Chancery and by the Board of Trade, and the matter was now finally settled.

He also announced that the Council had approved the formation of a LANCASHIRE AND CHESHIRE BRANCH of the Institution with Headquarters at Manchester. There was a proposition to found a WESTERN COUNTIES BRANCH, with Headquarters in Bristol, a Meeting in connection with which would be held in Bristol on 6th January.

The following members were elected in accordance with Article 14 of the Articles of Association :—

Members.

CADDELL, GEORGE, 59, Hulme Hall Lane, Newton Heath, Manchester ; *GEEN, THOMAS PERCIVAL, A.M.Inst.C.E., "Linden," Ty-Gwyn Road, Pen-y-lan, Cardiff ; GENEVE, LOUIS FREDERICK CHARLES ANTHONY, B.Sc., A.M.I.Mech.E., 111a, Holland Road, Kensington, W.14 ; GODFREY, EDWARD, M.Am.Soc.C.E., Monongahela Bank Building, Pittsburgh, Pa., U.S.A. ; LOBBAN, CHARLES HENRY, B.Sc., A.M.Inst.C.E., 35, Bloomsbury Square, W.C.1 ; MACKENNA, JAMES, 28, Castle Street, Swansea ; ROUSSEAU, Major MAURICE CHARLES, Royal Air Force, Halton Camp, Wendover ; WEBBE, HENRY JOHN ERNEST, 149, Nelson Road, Hornsey, N.8 ; WHITE, HARRY ROWLAND, 36, New Broad Street, E.C.2.

* Transferred from Student.

Associates.

HANBY, GEORGE ASHLEY, Technical College, Hull ; HENSER, FREDERICK GEORGE, 16, Adelaide Road, West Ealing, W.13 ; *JONES, ERNEST, 3, Whitehall Road, Handsworth, Birmingham ; LUYKEN, HENRY MARTIN, A.R.I.B.A., Hotel Clieveden, Westcliff-on-Sea ; § TELLERY, FRANK IGNATZ, A.R.I.B.A , M.S.A., c/o Alliance Bank of Simla, Rangoon ; WYNNE-JONES, NEVILLE, A.R.I.B.A., M.S.A., M.J.I.E., P.W.D., Colombo, Ceylon.

§ Transferred from Associate-Member.

* Transferred from Graduate.

Associate-Members.

*HYPHER, FRANCIS ALOYSIUS, B.Sc., M.R.San.I., c/o Bengal Nagpur Railway, Kidderpore, Calcutta, India; *NIELSON, WALTER KENNEDY ROBBIE, 126, North Frederick Street, Glasgow; *OLIVER, EDWARD AMBROSE, 88, Golden Hillock Road, Small Heath, Birmingham; RUANE, AUSTIN ANTHONY, 75, Pilot Bunder Road, Colaba, Bombay, India; *STARKEY, LEONARD JOHN, "Ingestre," Cannon's Lane, Pinner.

*Passed Examination.

Graduates.

§*GILCHRIST, JOHN ALBERT, 13, Annie Street, Seedley, Manchester; *GRANT, ERNEST, 95, Bridge Road, Itchen, Southampton; *MURTAZA, SAYYID GHULAM, c/o Messrs. Henry Adams & Son, 60, Queen Victoria Street, E.C.4; *NICHOLLS, BRUCE JAMES HURMAN, 185, Felsham Road, Putney, S.W.; WOOD, HORACE, 131, Tonge Moor Road, Bolton, Lancs.; *WRIGHT, SAMUEL THOMAS, 19, Gaskarth Road, Balham Hill, S.W.

§ Omitted by printers' error from last list of elections.

* Passed Examination.

Death.

Regret was expressed at the news of the death of ALEXANDER HALL, Member, of Egypt.

Resignations.

The Secretary reported the acceptance by the Council of the following resignations, most of which were due to retirement from the profession :— ADDISCOTT, H. H., Member, of London; BROUSSON, R. P., Member, of London; BULL, T. H., Member, of London; CAWLEY, G., Member, of London; CROTHALL, H. G., Member, of London; DEACON, T. M., Member, of London; GADD, W. L., Member, of Gravesend; HINDSLEY, J., Member, of London; HUGHES, J. C., Member, of London; GOAD, C. E., Member, of London; LESLIE, P. W., Member, of London; LLOYD, R., Member, of Paris; MARSH, C., Member, of London; MARTIN, V. J., Member, of Cheshire; MORGAN, F. E., Member, of India; REED, G. H., Member, of Wadebridge; SAISE, W., Member, of Bristol; SELBY, C. R., Student, of London; ULDALL, S. V. A., Member, of Hong Kong; WHITAKER, J., Member, of Manchester.

Mr. A. INGERSLEV, B.Sc., then read a paper on "The Strength of Rectangular Slabs." A discussion followed, and a hearty vote of thanks to the Lecturer, moved by Mr. E. S. ANDREWS and seconded by Mr. W. A. GREEN, was carried with acclamation.

Mr. INGERSLEV replied to the vote of thanks and the discussion, and the proceedings terminated.

EXAMINATIONS.

The next Examination will take place on 10th and 11th May, 1923. See p. iv. Advertisement Section.

REINFORCED CONCRETE

Government Insurance Offices, Brisbane, Queensland, Australia. For The State Government.
Government Architect: G. G. Hutton, Esq. Floors reinforced with B.R.C. Fabric.

B.R.C. Fabric is made from steel in its strongest form for structural purposes, fifty per cent stronger than ordinary rolled steel. The wires are accurately and permanently fixed beforehand and the Fabric arrives on the job ready for simple laying. Ease and accuracy of construction are secured with this regular high quality material, and out of them comes strength and safety. Columns and Beams reinforced with B.R.C. Hoops, Helicals and Grouped Stirrups made by machine and delivered ready for fixing have the same high degree of correct workmanship. Buildings reinforced with B.R.C. Fabric are the strongest of all reinforced concrete constructions.

The BRITISH REINFORCED CONCRETE ENG. Co. LTD.

Head Office: 1 DICKINSON ST., MANCHESTER *Works:* TRAFFORD PARK, MANCHESTER
Branch Offices: LONDON, Iddesleigh House, Caxton Street, Westminster, S.W.1;
LEEDS, 3 Park Lane; LIVERPOOL, 36 Dale St.; BRISTOL, 47 Whiteladies Road;
GLASGOW, 62 Robertson Street; DUBLIN, 12 Ely Place;
BELFAST, Mayfair Buildings, Arthur Square

BRC
Fabric

Telegrams:
"Anglodany, Estrand, London."

Telephone:
CENTRAL 6138.

Anglo-Danish Concrete Construction Co.,

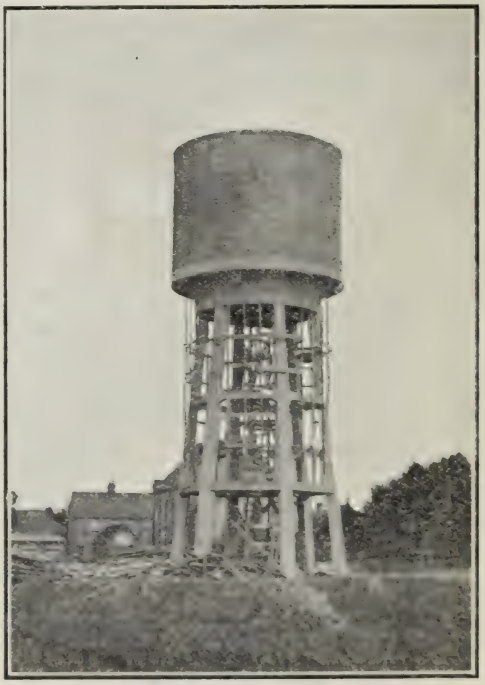
LIMITED,

7/8, NORFOLK STREET, W.C. 2.

ENGINEERS & CONTRACTORS

SILOS .. WATER TOWERS .. BUNKERS
CHIMNEYS .. FACTORY BUILDINGS, &c.

Sketch Proposals Free.



Water Tower at Selsey, nr. Chichester.
Capacity, 50,000 galls.

INDUSTRIAL CONSTRUCTIONS LIMITED

DESIGN AND ERECT
ALL TYPES OF
REINFORCED CONCRETE
STRUCTURES.

TELEGRAMS: "Refcretcon, Phone, London."
TELEPHONE: Victoria 6590 (3 lines).

44 GROSVENOR PLACE
LONDON :: :: S.W.1.

SIMPLEX CONCRETE PILES

PILE FOUNDATIONS and
REINFORCED CONCRETE
STRUCTURES of all descriptions.

Our method of constructing and driving Simplex Concrete Piles at one operation obviates weeks of delay in commencing and completing work.

Write for Illustrated Booklet:

SIMPLEX CONCRETE PILES, LTD.,
104, VICTORIA ST., WESTMINSTER, LONDON, S.W.1

Works: EDMONTON.
Scottish Office: 10, COOK STREET, GLASGOW.

ALL WHO ARE INTERESTED IN CONCRETE

should pay a visit to THE

Permanent EXHIBITION

of Concrete Products

AT

143, Grosvenor Road, S.W.1.

(Near Vauxhall Bridge.)

OPEN DAILY (Saturdays excepted) 10 to 4.

This collection includes examples of the use of concrete in the following interests:

RAILWAYS, ESTATE & FARM,
MUNICIPAL AUTHORITIES,
ARCHITECTS & BUILDERS

and others.

This Exhibition has been arranged by the

CONCRETE UTILITIES BUREAU

35, GREAT ST. HELENS, LONDON, E.C.3.